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Mr Arthur Floyd, Member, Chairman of the Division, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Author.

Road Paper No. 49

THE PLANNING OF RING ROADS, WITH SPECIAL REFERENCE TO LONDON

by

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SYNOPSIS

Ring roads in or around towns frequently figure in planning proposals and the Paper considers their purpose, traffic and economic values, and practicability.

Part I develops a theoretical approach, and Governmental views and their inconsistencies are summarized. Some ring roads, or parts of them, have been constructed and comments are made on them; brief reference is also made to similar proposals in current development plans in Britain. The basic principles of design are discussed in detail. Distinctions are drawn between "through" and other groups of traffic likely to use ring roads. Factors affecting the probable use of the roads are summarized and the importance of relative speeds is emphasized. The main theoretical conclusions are summarized.

Part II cites various proposals for ring roads in London from that of Paxton a century ago, to those in the County of London Plan in 1943. Reference is made to public information on the proposals for a ring motorway round central London and to the Government's decision that it should not be constructed.

Part III examines the case for the road. The nature, volume, and movement of traffic in central London are described, together with post-war surveys on destinations of traffic entering by the radial roads and the average speeds in the central area.

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An analysis of the potential use of the ring motorway is made and the effect on traffic conditions in the central area is discussed.

The design of the ring road, its curves, gradients, widths, lighting, sections in tunnel, and layout of access points is discussed, but constructional considerations are referred to only in general terms.

The construction programme is considered from the aspect of traffic value and practicability and the length and location of the minimum useful length are described.

Comparison is made between the ring road and ordinary street improvements which would provide the same traffic accommodation.

The economic value of the ring road is considered. Savings in time, fuel consumption, vehicle wear and tear, and accidents are some advantages, of which time is the most important. The gain to the community is assessed. Questions of amenity, rehousing, and legislation are touched upon, and it is concluded that such a project is practicable and valuable.

Part I

The theory of ring roads

Official views

Governmental views are contained in two post-war documents.[1, 2] The Built-up Areas Report suggests a single ring road for a town of 20,000 people and three or more for towns with a population exceeding 100,000. The primary function of an outer ring is to serve local traffic, but through traffic is also envisaged in certain circumstances. The need to compensate the greater distance to be travelled by higher speeds, increased convenience, and other advantages is recognized. The purpose of an intermediate ring road is to serve the partially developed area between the outer and inner ring and it is therefore likely to carry a greater variety of traffic. The inner ring is for local traffic only and its purpose is to relieve the small central area of the town of as much traffic as possible. If only one ring road can be constructed grade separation for it may have to be envisaged. The Planning Report deals primarily with inner rings but appears to contemplate a larger central area.

The above summary indicates that the views are stated in very general terms and a study of the Reports reveals certain inconsistencies, resulting from lack of definition in the object to be sought and a lack of detailed advice.

Existing examples

Most official views were probably based on existing examples. Town planners have favoured ring roads for more than a century and Washington, D.C., is an example of a ring/radial city planned in 1791, but most other American cities have a rectangular street pattern and ring roads are inapplicable. Many of the older towns in Europe have surface ring roads on the site of mediaeval or later fortifications. In Britain certain large towns, e.g., Birmingham and Manchester, have developed ring roads during the present century and others, e.g., London and Liverpool, have partly completed semi-circular roads.

Current proposals

Ring roads were a conspicuous feature of several of the so-called "master plans" formulated, usually on a regional basis, during and immediately after the 1939–45 war. Some were designed as fast motor roads with limited access points, but the majority were normal high-standard roads on which only moderate speeds would be

[1] The references are given on p. 123.

reasonably safe. The Plans for the County of London and for Greater London featured both types, but were not conditioned by time or resources.

The ring-road concept is still prominent in planning proposals. The Plans, being related to a period of 20 years and, in varying degrees, having regard to possible financial resources, are less ambitious and more realistic than the "master plans." In general, the ring-road proposals are confined to all-purpose streets encircling the central area although certain cities, such as Birmingham, Leeds, Leicester, Manchester, and Nottingham propose additional rings, either as new projects or as continuations of other proposals or existing sections of rings. The Author has been unable to trace any proposal for a motorway ring.

It is, however, interesting to note that of eight new town plans, which immediately preceded the development plan for existing towns, only one—Crawley—contains a ring road. This absence of ring roads is probably due to two main factors; first, these towns are not intended to be large (the maximum population envisaged being about 80,000), and secondly because, with the scope presented by almost entirely new building, the street plan can be so contrived that convenient arrangements can be made for through traffic in other ways. Another adverse factor may have been the system of financing the new towns. In the case of Crawley conditions are different. The new development adjoins the existing town and nearly half the ring road exists already in the shape of the Crawley By-pass on the London–Brighton road.

From a review of the rather indefinite official policy on the subject and references to the continued popularity of ring-road proposals in development plans, attention can now be directed to a theoretical analysis.

Basic principles

Any theoretical consideration must have a logical basis if the conclusions are to be sound, and there can be no disagreement with the general planning principle that it is desirable to relieve the busy central areas of towns of traffic which has no need or wish to go there. In the simple case of a by-pass for a village on a main road this aim meets with considerable success. The by-pass to Garstang on the Preston–Lancaster road attracted 87% of the traffic in 1929 and 85% in 1939, by which time total traffic had doubled. At Blackrod on the Wigan–Preston road, 90% was by-passed during the period 1935–1937. In the case of towns the proportion of traffic likely to use the by-pass tends to decrease as the population rises, e.g., at Dorking and Guildford in 1935 by-passes were taking 60% and 70% of the total traffic respectively, whereas at Shrewsbury and Oxford the proportion was about one-third. Reliable comparable information for larger cities in Britain is lacking, but an extensive investigation a few years ago in America showed that, whereas in towns of up to 10,000 inhabitants the percentage of by-passable traffic ranged between 51 and 43, at the other end of the scale the percentage range for towns with population between 250,000 and 500,000 was from 9 to 8, and in the case of larger cities with up to 1,000,000 inhabitants as low as 4. The comparable overall range in Britain, with shorter distances between towns, may be between about 80% and 10% or less. A ring road is, simply, a series of overlapping by-passes to various through radial roads and the extent of relief would be similar in each case. The inference is that in large cities a ring road is unlikely to be economically justifiable solely because of its ability to divert through traffic from the central area.

Through traffic, however, is not the only group of the total urban traffic to which a ring road would be attractive and the larger the town the higher will be the proportion of local traffic on any ring road.

It is, therefore, important to seek accurate definitions of traffic groups in relation to ring roads. It is usually possible in any large town to outline the developed area and the part of it in which traffic volumes exceed, and speeds fall below, certain arbitrary levels causing congestion and delay (Fig. 1). In any such urban area traffic can be divided into three groups, namely:—

(1) Vehicles whose origins and destinations are outside the area but which, in the absence of other facilities, would normally pass through it.
(2) Vehicles whose origins (or destinations) are outside the area and whose destinations (or origins) are inside it.
(3) Those whose origins and destinations are inside the area.

Convenient descriptions for these different groups are, respectively "through," "radial," and "internal local."

Fig. 2 shows the town with a ring-road alignment, and division into the same traffic groups is applicable to the area enclosed by the ring. Fig. 3 shows the normal and alternative ring routes for a vehicle in Group (1); Fig. 4 the alternative routes for the vehicles in Group (2) according to whether they would save time by using the ring

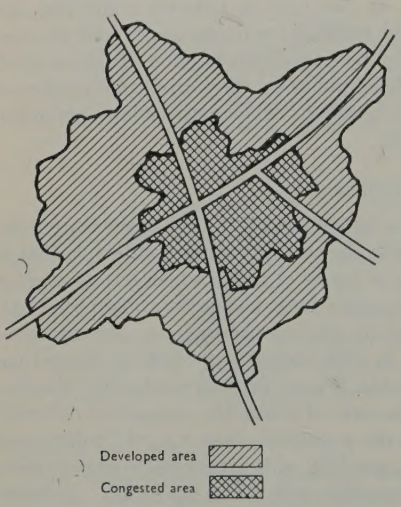

Fig. 1.

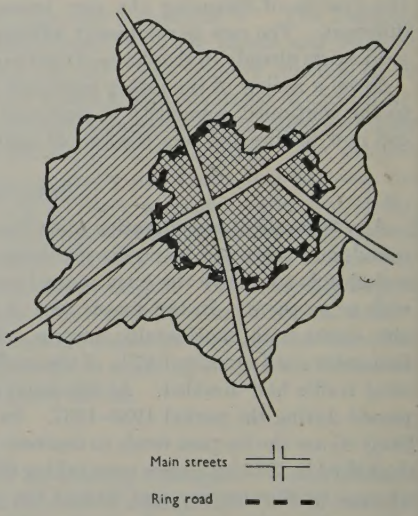

Fig. 2

or by proceeding directly to their destination; and Fig. 5 illustrates the alternative routes for internal local traffic according to whether time would be saved by using the ring or not. Each of the journeys illustrated could, of course, be made in the reverse direction.

Factors affecting the use of ring roads

There are many factors, such as ease of access and egress, location, amenity, and speeds, affecting the extent to which ring roads will be used.

Of these, speed is by far the most important. Assuming that the central area is enclosed by a circular ring road, the distance travelled by a vehicle using the ring

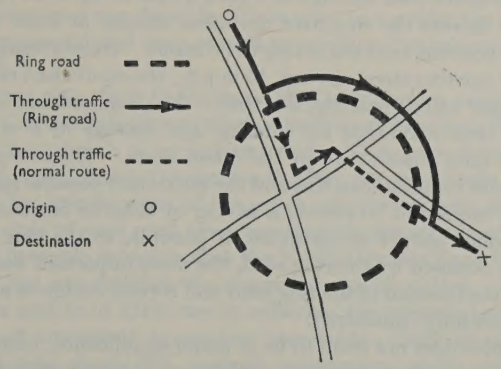

FIG. 3.—SPECIMEN JOURNEYS—THROUGH TRAFFIC

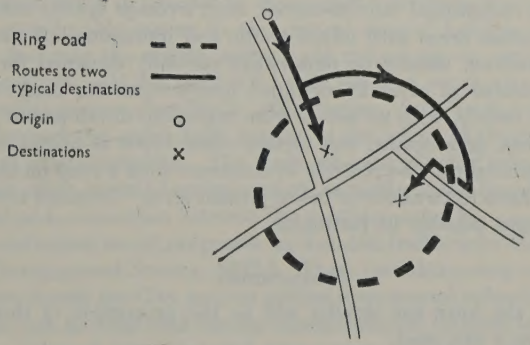

FIG. 4.—SPECIMEN JOURNEYS—EXTERNAL TO INNER AREA

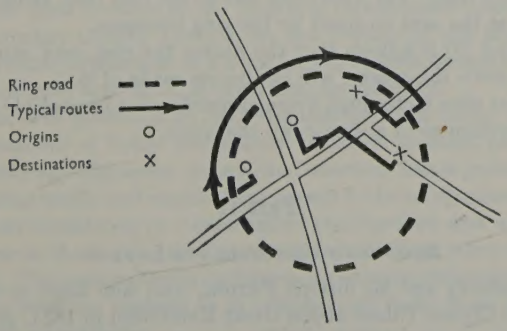

FIG. 5.—SPECIMEN JOURNEYS—INNER CIRCLE

road instead of the direct road through the town would be up to 50% longer. To be attractive to most drivers the ring-road deviation should, at least, show no loss of time and should, if possible, enable a saving to be made. If the average speed through the town centre by existing streets is, say, 15 m.p.h., the equivalent ring-road journey should take no longer and, if possible, less time. The required average speed on the ring road, with a time allowance for entering and leaving it, is a little less than 32 m.p.h., or more than double that on the direct route. The average speed on the ring road itself should be more than this and the maximum possible speed still higher.

It is even more important to enable a saving of time to be made for the other groups of traffic which should be attracted, if possible, to the ring road. Finally, the larger the area enclosed by the ring road, the more important becomes the question of speed, and the location of the ring road and correct design of access and egress points should be carefully considered.

The amenity aspect does not seem to be of major significance, because the number of vehicle journeys on business greatly exceeds the number on pleasure, and business journeys are far more important economically.

Except on short ring roads either in a large town enclosing a small central area with no through-traffic facilities or in small towns, average speeds on the ring should be at least 30–35 m.p.h., preferably more but, however high the standard of road design, except in exceptional circumstances, such average speeds cannot be maintained safely in urban areas with mixed traffic and numerous intersections. Ring roads should, therefore, usually be motorways specially designed for high speeds, with a limited number of carefully designed intersections for traffic entering and leaving the main radials, with no access from adjoining development and with the exclusion of cyclists, pedestrians, and certain other types of mixed or slow traffic. It will not, as a general rule, be possible to construct such a road on the surface and it will, therefore, have to be above or below ground level. In small towns ring roads on the surface might possibly be justifiable.

Conclusions

(1) The larger the town the smaller will be the proportion of through to total traffic likely to use a ring road.

(2) Radial traffic will form the largest group on the ring road, but, because such traffic becomes heavier as the centre of a town is approached, the attraction of the road to such traffic decreases as the area enclosed by the ring increases.

(3) Internal local traffic will contribute less to the ring than radial traffic and will tend to decrease as the area enclosed by the ring increases.

(4) From (2) and (3) it follows that the nearer the ring road can be sited to the central area of a town the greater will be the use made of it.

(5) Owing to the time factor high average speeds must be possible on the ring road and it should therefore be of a motorway character.

Part II

Ring-road proposals for London

More than a century ago Mr Joseph Paxton, who won fame as an engineer and architect with his Crystal Palace at the Great Exhibition in 1851, put forward plans and estimates for a ring road 72 ft wide, in a covered arcade, flanked by houses and shops and with an eight-track railway above it. It was to have been 11·5 miles long,

roughly elliptical in plan, and with special new bridges at Rotherhithe and Westminster. The Committee appointed by Parliament to consider the traffic difficulties of the day were impressed and gave partial approval to the scheme, but Parliament decided against it.

Although not true ring roads in the sense that they form a continuous route, the North and South Circular Roads nevertheless fulfil some functions of a ring and form the earliest of such proposals to be implemented. They were recommended by the Greater London Arterial Roads Conference in 1916, not as a single ring, but as two separate projects for linking some of the outer suburbs on both sides of the River Thames, and the ends of each road are a considerable distance apart, particularly in the west. There is ample evidence that the North Circular Road has been of great value, even if it has not fulfilled the designers' special object. It has, however, serious limitations and is of little use in relieving the central London area. The route falls short of consistent or modern standards. Only about 20% has dual carriageways and only about 15% a single carriageway width greater than 30 ft. Several sections are narrow, with sharp turns and other unsatisfactory features, and about 18% is subject to speed limits. Nevertheless, car tests in 1936 gave an average speed of 23·6 m.p.h. over a length of $22\frac{3}{4}$ miles.

Less progress has been made with the South Circular Road which was planned to run from Woolwich Ferry in the east to Wandsworth Bridge in the west. Generally this route lies considerably nearer to the central area of London than the North Circular Road, but only the eastern section—$4\frac{1}{2}$ miles long—is complete to more or less modern standards. Of this section 1·1 mile has dual carriageways and 2·3 miles a carriageway width greater than 30 ft. The portion of this section subject to speed limit is 2·6 miles and the speed limit applies to the whole of the rest of the route, which consists of existing streets. The route, in spite of being incomplete, is useful in diverting from south central London part of the traffic to and from the Kent radials and forms a valuable connexion between the southern suburbs.

The next co-ordinated set of proposals for London traffic were those contained in the Highway Development Survey, 1937.[3] These included a ring road in the form of a rough ellipse round the City and an orbital road several miles farther out in the Home Counties, but no ring road for the central area of London as a whole. The interconnecting series of improved and new routes (including the South Circular Road) can hardly be considered as forming a ring road.

In 1941 the City Corporation authorized the preparation by its engineer Mr F. J. Forty of a plan for the redevelopment of the City. This[4] contained a ring-road proposal on similar lines to those of the Highway Development Survey.

In 1941 the London County Council, at the request of the Minister of Works decided to entrust the preparation of a Plan for the redevelopment of the County to Sir Patrick Abercrombie and Mr J. H. Forshaw, its architect.

Before the Plan[5] was published in 1943, the late Sir Alker Tripp, at that time the Assistant Commissioner of Police responsible for traffic matters, proposed an inner ring road of a motorway character, constructed above or below ground level in order to eliminate crossing traffic and enable high speeds to be safely attained. Although this suggestion was considered by Abercrombie and Forshaw and was supported by the Chief Engineer to the L.C.C., the County of London Plan when it was published did not include it. The reasons for its exclusion can only be conjectured, but the effect on amenities of a road on viaduct or in cutting may have been regarded as outweighing the traffic advantages.

In 1944 the Planning Committee of the Royal Academy, under Sir Edward Lutyens,

Sir Giles Scott, and Sir Charles Bressey, proposed an inner ring road, largely in cutting, and of a motorway character. This showed that some eminent architects, at any rate, did not share the amenity doubts which had been expressed in other quarters about a sunken motorway round the central area of London. Sir Patrick Abercrombie, in an article in "The Times" in 1951 on the County Development Plan, wrote that the project "respected planning requirements and was brilliantly detailed, but its cost was prohibitive."

In considering the road proposals of the County of London Plan the background should be borne in mind. The plan was not an L.C.C. plan, but one formulated in a consultative and advisory capacity by its Authors, who made clear that they had conceived their instructions in the widest terms, assuming that new legislation and financial assistance would be forthcoming. In other words, their proposals were not related to any specific period or to the money, material, and labour resources available. To engineers such an approach lacks realism.

The road proposals of the Plan were based on the ring/radial principle, supplemented by cross-routes (Fig. 6, Plate 1). Three ring roads were proposed. The main entering radials were to be converted into motorways as far as the middle ring, and five were to be extended to the central area as two cross-routes with an exceedingly complicated intersection in the Aldwych region. The innermost ring (referred to as the "A" ring) was to enclose the central area of greatest traffic density, but was to be of normal high-standard surface type. The next ring outwards ("B") was to be a motorway for fast traffic and one of its main objects was stated to be "a main route for industrial traffic between the docks and the western exits." The third ring ("C") was to incorporate the North and South Circular Roads and to include considerable lengths of new road to connect them. As if rather doubtful of the extent to which the "B" motorway ring would achieve its objects, the two cross-routes, partly in tunnel and elsewhere virtually of motorway standard, were also included.

The Authors of the Plan appear to have been largely influenced in the choice of position of the fast ring road by the facts that the districts through which it would pass were less densely developed than those through which the "A" ring would be located and that parts of those areas should have first priority in post-war reconstruction on account of war damage and obsolescence. Traffic needs seem to have been secondary. The "B" ring, they claimed, "would serve as a by-pass to central London, warding off the through traffic some distance away." The Plan gave no estimate of the proportion of such traffic to the whole and its Authors, despite engineering advice that it would be small, seem to have assessed it as high. The same over-emphasis was given to dock traffic. The proposal for a motorway type of "A" ring had been very carefully examined said the Authors, but they believed that, taking all matters into consideration, the motorway ring should be in the "B" position and that this and the major cross-road system would provide a more comprehensive and satisfactory solution.

The L.C.C., when considering the County of London Plan proposals in July 1945, were no doubt influenced by the facts that the police favoured a motorway on the "A" alignment and that engineering advice was to the effect that such a proposal was sound and practicable. The Council accordingly resolved that the main highway structure of London should be based on the conception of a series of ring roads "A," "B," "C" and a series of radial roads linked to the "A" and "B" rings. Wherever possible new main traffic routes should be planned so as to create the minimum disturbance of existing communities. No reference was made as to whether any of

these rings were to be of a motorway type and it can be inferred that the divergence of views on the location of such a motorway ring had not been resolved at that time.

Some indication of the subsequent history of this project is given by answers to questions in the L.C.C. Proceedings, which referred to discussions between the Council and the Ministries of Transport and Town and Country Planning with a view to determining the character of the "A" and "B" rings.

No further public announcement was made until the 2nd May, 1950, when the Minister of Town and Country Planning, replying to a question in the House of Commons whether he could announce the Government's intentions in regard to the "A" ring road, said that this project would not now proceed, but that other plans for road improvements in London would be studied by the Ministry of Transport in consultation with himself and the L.C.C. No reasons were given for the abandonment of the proposal, but on the following day "The Times" stated, presumably on information supplied by the Ministry of Town and Country Planning, that the considerations on which this decision had been based were understood to have been:—

(1) That the proposed ring road would confer little, if any, benefit until one-third had been completed—a task of some years' duration.
(2) That it would entail disrupting the existing street plan and substantial demolition work which would leave ugly scars pending redevelopment.
(3) That it was estimated that the cost of the engineering work alone would have been about £88,000,000, which it was felt could be more usefully spent in improving the existing street pattern.

Whatever the reasons, it was evidently clear to the L.C.C. that neither this nor any other similar single project would be acceptable to the Government because nothing of this individual magnitude was included in the Council's Development Plan.[6]

In 1951 the London and Home Counties Traffic Advisory Committee[7] informed the Minister of Transport at that time that it viewed with concern the decision to abandon the "A" ring project. Finally in April 1955 it was stated in the House of Commons that the Minister of Transport and Civil Aviation was considering the possibility of reviewing the proposal, which, he had been advised, would be a very valuable aid to the relief of traffic congestion in London.

Part III

The case for a ring road in London

Nature of traffic in inner area

Volume.—The inner area of London for the purposes of this Paper is approximately that enclosed by the innermost two rings in Fig. 7, Plate 1. Before any estimate could be made of the effect of a ring road on the traffic in this area it was necessary to consider its volume, composition, lengths of journeys, and speed. From 1937 data it was calculated that between 9 a.m. and 6 p.m. on a normal week day an average of 18,000 vehicles (excluding bicycles) entered the area hourly by the main routes, with a small additional number by secondary streets. A similar average number left each hour.

Volume, i.e., the flow of traffic in an area, is best expressed in vehicle-miles in an average hour. There are 70 miles of main streets in the inner area and, on provisional assumptions that through traffic was 20% of the total entering, that the mean journey distance of radial traffic was 2 miles, and that a similar distance for

the internal traffic of, say, 10,000 vehicle-journeys per hour was 1½ mile, the average number of vehicle-miles per hour was estimated to be:—

Traffic group	Number of vehicles		Journey distance: miles		Vehicle-miles/hour
Through . . .	20% of 18,000	×	3·5	=	12,600
Radial inwards .	80% of 18,000	×	2·0	=	28,800
Radial outwards .	80% of 18,000	×	2·0	=	28,800
Internal . . .	10,000	×	1·5	=	15,000
					85,200

This estimate was checked by other methods and the totals were closely similar. On the 1954 police census data[8] the total traffic at twenty points on the fringe of the inner area showed an increase of 22·4% above the 1937 figures and if the 18,000 vehicles estimated as entering the area in 1937 and internal journeys are increased proportionately, say by one-fifth (to allow for certain variations in traffic behaviour), the volume in vehicle-miles per average hour on the main streets becomes, say, 102,000. Since the war the Road Research Laboratory has made a series of surveys on 34 miles of main streets on which the number of vehicle-miles was estimated, and, extending these figures to cover the inner area and allowing for the growth of traffic, the results were closely comparable.

Composition.—Taking the same twenty census points as the most reliable guide to the composition of traffic, the various classes of mechanically propelled vehicles, expressed in percentages, were as shown in columns 2 and 3 of Table 1.

TABLE 1

Class of traffic	1937 %		1954 %		1948 Sample: %	R.R.L. area 1954: %
Light commercial	21·4	28·8	25·9	31·3	33·5	33
Heavy „ . . .	7·4		5·4			
Cars, taxis, and motor cycles	53·5		55·1		51·2	57
Coaches	1·2		1·2		1·5	1 (?)
Buses, trams, and trolley buses	16·5		12·4		13·8	9
	100·0		100·0		100·0	100

For comparison, the sample survey composition is given in column 4. In the Road Research Laboratory survey of 1954 the composition was as shown in the last column but it should be remembered that this relates only to 6 hours of the working day, excluding peak periods, and to a rather different area.

Lengths of journeys.—The lengths of journeys were obviously an important factor in calculating the effect of a ring road and there was a considerable amount of uncoordinated data, of which the following are examples:—

(1) A survey in 1928 of traffic using Bond Street showed that only 10% traversed the whole street.

(2) The Highway Development Survey made a destination survey of traffic

leaving the London Docks. This showed that at a radius of $1\frac{1}{2}$ mile no road was carrying on an average more than thirty vehicles/hour from the docks, but that close to the docks the proportion was much higher (e.g., 215 vehicles/hour in Commercial Road East).

(3) A survey in 1938 at Woolwich Ferry showed that the origin or destination of 70% of the vehicles crossing were less than 5 miles away.

(4) Analyses of traffic movements at intersections almost always show that a large proportion of the traffic turns left or right.

Weighing all available evidence it was concluded that many vehicles made relatively short journeys and that only about 15% of the vehicles circulating in the inner area was through traffic.

Speeds.—Potential speeds on the ring road compared with average speeds in the inner area would greatly influence the probable use and the economic effects of the ring. In 1948 there was a considerable amount of fragmentary data on this important subject.

In 1936 test runs by a 16-h.p. car were made for the Highway Development Survey during working hours (8 a.m.–7 p.m.) on normal weekdays on three selected routes. Outside the inner area average speeds of up to 18 m.p.h. were reached, but in the City they fell to 5·85 m.p.h. and between Euston Road and Trafalgar Square to 7·7 m.p.h. Over all three routes the average was 12·5 m.p.h. but it was less in the inner area sections. In 1948 a 10-h.p. car on the same routes averaged 12 m.p.h. in the inner area, but on the two sections with the lowest average speed in 1936 50% higher speeds were recorded. The substitution of trolley buses for trams on some streets, slightly lower traffic volumes generally (but only half the number of pedal cycles), and more extensive use of signal control at intersections were among the causes of this improvement.

This information was not considered adequate and in 1948 a series of tests was made to find the average speed of private cars in the inner area between 8 a.m. and 8 p.m. An experienced driver with a 10-h.p. car proceeded from one point to another picked at random, choosing his own route, driving with care, and observing the speed limit. A large number of these journeys were made, an observer noting the route chosen, the time taken, and the distance. The analysis showed that 82·5% of the mileage was on Class I and II roads. The average speed on various sections varied between 8 and 25 m.p.h. and over all the routes was about 15 m.p.h. during 12 hours and 13·5 m.p.h. between 9 a.m. and 6 p.m. There were variations between different hours of the day, following very closely changes in volume.

To compare the average speeds with the time taken on a reasonably direct route such as a ring road, the straight-line distance of all journeys was measured and found to average 71% of the actual distance. For the 12-hour day the equivalent straight-line speed was 10·8 m.p.h. and for the period 9 a.m. to 6 p.m. 9·6 m.p.h. On the 1936 routes the equivalent straight-line speed was 10·6 m.p.h. For the purpose of this Paper an average speed for all vehicles of 10 m.p.h. on a straight line has been taken for the inner area throughout the working day.

Further journey investigations

The other factor on which adequate data was lacking was the nature of journeys of vehicles entering the inner area and to provide more information a special survey was made in 1948 in two stages, namely:—

(1) A census of the numbers and types of all vehicles and of destinations of private cars and commercial vehicles entering the area by seven main

streets (Grosvenor Road, Bayswater Road, Hampstead Road, Commercial Road, Old Kent Road, Kennington Road, and Upper Street, Angel) between 8 and 10 a.m.

(2) A check on two of these streets (Grosvenor Road and Commercial Road) between 11 a.m. and noon and between 3 and 4 p.m. to establish the relation between off-peak and peak conditions.

The census points were as close as practicable to the proposed alignment of the ring road. Of the drivers questioned 94% replied, and their vehicles constituted 76·6% of the total of 9,530 which entered the area in 2 hours by the streets surveyed, a reasonable sample of all vehicles entering the area.

The destinations of all known journeys were then plotted (Fig. 6, Plate 1, shows one example) and their direct-line lengths measured. This analysis showed:—

(1) That 75% of the destinations lay between 0·5 and 3·5 miles from the point at which the vehicle was intercepted and that only 10% exceeded 5 miles. This confirmed the view that most journeys were short.

(2) In order to estimate what proportion of these journeys could be made more quickly by the usually longer but faster ring road than by ordinary streets, areas were plotted in accordance with known average point-to-point speeds, and with allowances for the time taken to enter and leave the ring road, within which destinations could be reached more quickly by the latter route (Fig. 8). From the destination plots between 47 and 52% of

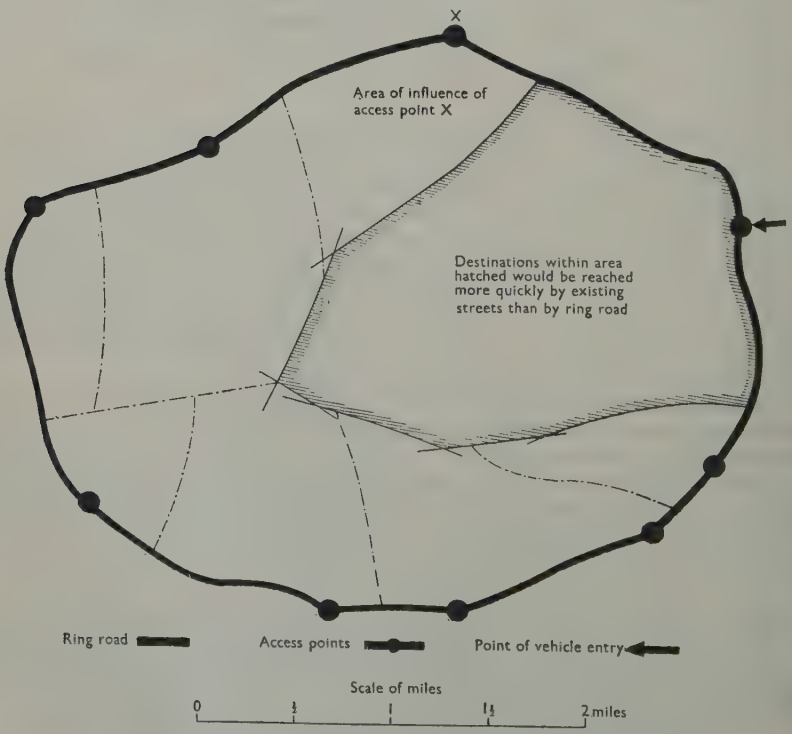

FIG. 8.—TYPICAL "AREA OF INFLUENCE" JOURNEY DIAGRAM

the vehicles entering the area enclosed by the ring road and whose destinations were within it would save time by using the ring instead of the direct route. It was considered reasonable to adopt 50% as a general factor.

(3) About 20% of the total counted was proceeding to destinations beyond the area enclosed by the ring road, but a small proportion was unlikely to use it. Taking into account the proportion of eligible vehicles (e.g., taxis) not questioned and the effect of the siting of the census points, it was decided, on balance, that the earlier estimate of 15% for through traffic was reasonably accurate.

Potential ring-road traffic

The next stage was to estimate the probable extent to which the ring road would be used. First, the classes of vehicles eligible to use the ring road were considered. Horse-drawn and other slow vehicles and pedal cycles should obviously be excluded, but some mechanically propelled vehicles would not wish to use it (e.g., buses with frequent stops) and others (e.g., motor-cycle combinations and motor-assisted pedal cycles) should be excluded in order to maintain high average speeds.

Next, taking the police data for 1954 as providing the best indication of composition and dealing with the proportions of different classes of traffic in more detail than previously, but ignoring the slow traffic referred to, the proportions of vehicles eligible for the ring road become:—

Class of traffic	Ineligible %	Eligible %
Light commercial	—	25·9
Heavy „	—	5·4
Cars and motor cycles	—	41·4
Taxis	—	11·7
Motor-cycle combinations	1·2	—
Motor-assisted pedal cycles	0·8	—
Coaches	—	1·2
Buses	12·4	—
	14·4	85·6

When applying the destination data obtained from the 1948 census in respect of commercial vehicles, cars, and motor cycles it is necessary to estimate the percentage in respect of taxis and coaches, whose drivers were not questioned. Taxis normally make short journeys and the proportion using the ring road might be one-third. The destinations of most coaches are in the central part of the inner area. The proportions of the various classes of eligible traffic likely to use the ring road expressed as percentages of all mechanically propelled vehicles (except tractors and locomotives) therefore become :—

Traffic group	%		%
Light commercial	25·9 × 50	=	12·95
Heavy „	5·4 × 50	=	2·70
Cars and motor cycles	41·4 × 50	=	20·70
Taxis		say	3·85
Coaches		„	0·20
			40·40

The estimated equivalent on the 1937 police census data was 37% and the actual equivalent on the seven main routes surveyed in 1948 was 40%. The trend being upwards, 41% is taken for the purpose of this Paper. The number of vehicles entering and leaving the ring road in an average hour is thus 41% of 21,600, i.e., 8,856. This figure is not, of course, the same as the flow on the ring road.

To estimate this flow expressed in vehicle-miles/hour, a calculation similar to that given on p. 107 may be made. Through traffic would be unaltered in volume but would travel farther. The volume of radial traffic would be less; from the 41% of eligible traffic for the ring road, the 20% representing through traffic should be deducted, and hence 21% of radial traffic should be taken. It would consist largely of that making longer journeys than the average 2 miles taken for the inner area. The distance is further increased by the greater length of journey on the ring road. In an average hour the ring-road traffic would thus be:—

Traffic group	Number of vehicles		Journey distance: miles		Vehicle-miles/hour
Through	20% of 21,600	×	4·0	=	17,280
Radial inwards . . .	21% of 21,600	×	2·9	=	13,154
Radial outwards . . .	21% of 21,600	×	2·9	=	13,154
					43,558

If 10% is added for peak hours, the flow becomes, say, 48,000 vehicle-miles/hour.

Extent of relief to inner area afforded by the ring road

Again using the basis on p. 107 and bearing in mind that the proportion of radial traffic attracted to the ring road would be that making longer than average journeys and that allowance has to be made for the journeys of such traffic between the ring and its destination, gives:—

Traffic group	Number of vehicles		Journey distances: miles		Vehicle-miles/hour
Through	20% of 21,600	×	3·5	=	15,120
Radial inwards . . .	21% of 21,600	×	2·5	=	11,340
Radial outwards . . .	21% of 21,600	×	2·5	=	11,340
					37,800
Deduct:—					
Radial traffic between ring and its destination:—	2 × 21% of 21,600	×	0·6	=	5,400
				Net relief	32,400

On p. 108 the volume of traffic on main streets in the inner area is taken as 102,000 vehicle-miles in an average hour and the relief afforded by the ring road would therefore be 31·8%. Neglecting any change in distribution of traffic this would mean that on 1954 volumes the inner area could accommodate at least 46% more traffic after the ring road had been constructed.

Relation between minimum average speeds on, and capacity of, the ring road

The minimum average speeds required on the ring road are different for through and radial traffic. The actual speeds expected under peak conditions of load have a fundamental bearing on capacity and, therefore, on design.

First, there is the case of through traffic with the minimum average speed equivalent to the straight-line speed of 10 m.p.h. in the inner area. The ring road would be elliptical not circular in plan (its main axes being 3·75 miles and its length slightly less than 3 miles) and it would therefore provide the least favourable alternative route for traffic on the short axis. The problem is best solved by considering two journeys between access points as planned (see Table 2).

TABLE 2

Section	Distance: miles			Journey time: min		Minimum average speed required on ring: m.p.h.
	By existing streets	Straight line	By ring	Straight line at 10 m.p.h.	By ring allowing 1 min for entering and leaving	
Tottenham Court Road –Old Kent Road .	3·75	3·1	5·1	18·6	17	18
Angel–Kennington Park Road . . .	3·0	2·9	4·8	17·4	16	18

For journeys on the longer axis the minimum average speed required would be less than 18 m.p.h., but much higher speeds could be safely observed and hence considerable time would be saved by through traffic.

The minimum speed requirement is, however, different for the considerably larger group of potential users comprising vehicles proceeding to or between points near the ring road but in the inner area. In the case of vehicles outside the inner area proceeding to destinations, say, 0·5 mile from an access point on the opposite side of the ring road and within the inner area, the journey times by the ordinary street system applied to the examples in the preceding paragraph would be 3·1 − 0·5 and 2·9 − 0·5 miles at 10 m.p.h., or 15·6 and 14·4 min respectively. Deducting one minute for entering and leaving the ring road and 3 min for the end of the journeys by the existing street system (0·5 mile at 10 m.p.h.) the equivalent journey times on the ring road would be 11·6 and 10·4 min respectively. The ring-road distances being 5·1 and 4·8 miles the minimum average speeds required would be 26·4 and 27·7 m.p.h. respectively.

From these examples two conclusions can be reached, namely,

(1) The higher the average speed on the ring road the greater will be the relief to traffic in the inner area.

(2) Coupled with the facts that average speeds increase outside the inner area and that most journeys are short, the larger the area enclosed by the ring road the higher should be the average speeds attainable if it is to attract traffic to and from the area enclosed.

In order to decide the number of traffic lanes and other basic design factors for the ring road and to estimate its economic value an average speed has to be assumed and,

although conservative for many vehicles, 30 m.p.h. has been taken for peak-hour conditions when the growth of traffic volume has nearly reached capacity. During the rest of the working day and with contemporary volumes 36 m.p.h. can be assumed. Both allow for curves, gradients, slightly lower speeds in the eastern tunnel section, a certain amount of weaving, and the mixture of commercial and private vehicles.

From p. 112 the peak-hour flow at present would be about 48,000 vehicle-miles/hour. The ring road being 11·1 miles long, the required capacity per mile per hour is (allowing for the variations in volumes of about one-eighth between different sections):—

$$\frac{48{,}000 \times 9}{11{\cdot}1 \times 8} = \text{say, } 4{,}900 \text{ vehicles.}$$

In one direction the capacity should be one-half, say, 2,450 vehicles/hour. If Clayton's speed/density curve [9] for a motorway is accepted, a lane at 30 m.p.h. (average) would have a capacity of 1,200 vehicles per hour. Two lanes would give double this, almost the required capacity; three lanes, except at access points where the ring-road traffic would be less than between them (for an example, see Fig. 9),

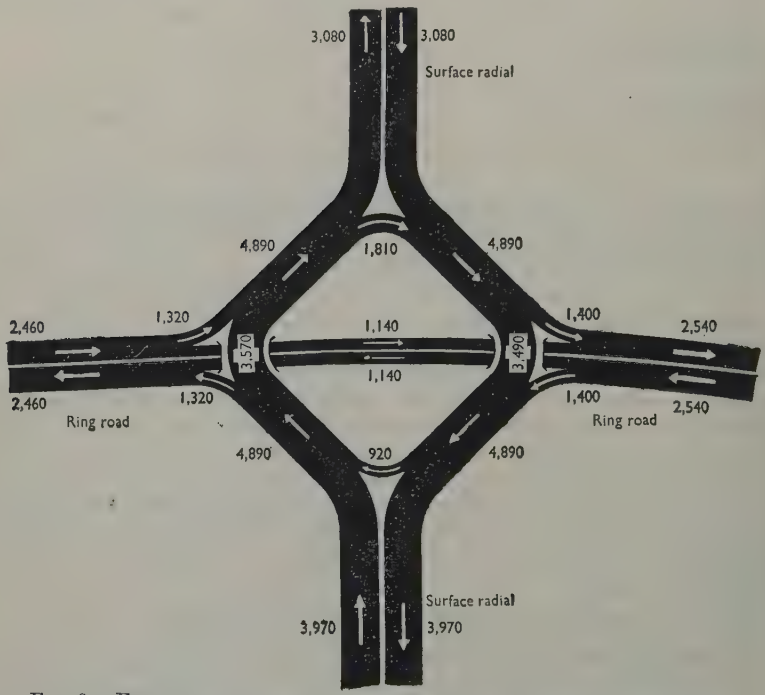

Fig. 9.—Estimated hourly volumes of typical access intersection
(Estimated bases:—1954 volumes; average working day 9 a.m.–6 p.m.

would thus give more than 45% reserve capacity. At an average 36 m.p.h. the capacity per lane would be 720 vehicles and three lanes would give a capacity of about 2,000 vehicles, i.e., nearly sufficient at this speed for the off-peak load. The maximum capacity per lane would be about 1,900 vehicles at 23 m.p.h., but at this

speed the ring road would be unattractive to many drivers. In short, traffic volumes and speeds on the ring road would tend to vary during the working day so as to maintain a rough equilibrium between the times required for journeys on it and on the ordinary street system.

Design

Grade separation.—In order to attain a moderately high average speed and to help in excluding slow traffic the ring road would have to be independent of the existing street network, except at a limited number of access points. The ring road must therefore pass over or under existing streets. Apart from the relative costs of elevated or sunken construction, several other factors have to be taken into account. For navigational reasons the river crossing near Tower Bridge must be beneath the Thames and for aesthetic reasons the section across Hyde Park below ground level. Generally elsewhere open sunken construction would probably be viewed with greater favour than viaducts, but a considerable section south of the Thames would run through low-lying water-bearing ground where it is possible that viaducts would be less costly. On the west the Thames would be crossed by a high-level bridge with three lanes in each direction. The greater part of the length north of the river would be sunken and the most important factor influencing alignment is the necessity to avoid interference with the Metropolitan Railway and important main sewers. On the sunken portions sloped banks rising from low retaining walls at driver's eye level and planted with grass, shrubs, and small ornamental trees should be adopted where possible in the interests of amenity and the avoidance of monotony to drivers.

Width.—It has been seen that generally three lanes in each direction on the sections between access points should be provided. Typical cross-sections are illustrated in Figs 10a to d, Plates 1 and 2. The standard lane adopted is 11 ft, with each roadway of three lanes separated by a central strip 6 ft wide. On each side of the road footways, each 3 ft wide, should be provided for patrolling or for use in an emergency. On curves the roadway widths should be slightly greater.

Tunnels.—The number of lanes in the tunnel sections required special consideration. At the crossing near the Tower Bridge, where twin tunnels are proposed to avoid the necessity of going too deep and so making the approaches inordinately long, it should be remembered that constructional difficulties might make a three-lane tunnel impracticable. Furthermore, however good the standard of lighting in a tunnel, some reduction in speed from that on an open road is inevitable and a rather lower average speed than 36 m.p.h. would have to be accepted. Under these conditions twin two-lane tunnels would provide adequate capacity. In the case of the Hyde Park crossing where, for amenity reasons, the surface must be reinstated, the approaches would be more or less at the same sunken level and no special constructional difficulties are expected, the form of construction would be twin roadways of rectangular cross-section. There is therefore no reason, apart from cost, why three lanes in each direction should not be provided on this section. It is considered that the extra cost would be justifiable.

Other basic factors.—Although the highest average speed used in calculations for capacity and economic estimates was only 36 m.p.h., the ring road was designed for a safe maximum speed of 70 m.p.h. This requires a mean minimum radius on the centre-line of 1,000 ft with transition curves and a maximum superelevation of 1 in 10. Generally, the steepest gradient was 1 in 30 (3·3%) but in one or two places railway crossings imposed a gradient of 1 in 20 (5%). On vertical curves a minimum of 1,000-ft radius was required for sight lines. The standard head room of 16 ft 6 in.

was adopted throughout. Lighting and ventilation of covered and tunnelled sections would have to be of high standards. The lighting would be graduated and controlled automatically by photoelectric devices to enable these sections to be entered in daylight without hesitation.

Access points.—The location and design of access points presented several interesting features. A balance had to be struck between too many and too few; too many would have increased the cost and too few would have reduced the traffic value of the ring road. The number eventually decided upon was nine, located primarily with a view to access to and from the principal traffic routes connecting with the trunk roads on the county boundary. Three (at Eaton Square, Montague Square, and the Angel) required short lengths of new surface streets to connect with existing radials. The others were to be at Tottenham Court Road, Aldgate (Gardiner's Corner), Abbey Street, Old Kent Road, Kennington Park Road, and Vauxhall Cross. It was important to design these intersections so that traffic movement should be as free as possible. First, it was necessary to ensure that additional traffic on the surface approaches and exits could be accommodated and existing improvement proposals were reviewed, and, where necessary, modified. Next, the problem of handling the considerable proportion of turning traffic on the surface had to be solved and the most suitable method appeared to be that of a surface roundabout. Four ingress and egress ramps, each of two lanes and about 700 ft long, to and from the ring road were required at each point. The detailed layout of a typical intersection is illustrated in Fig. 11, Plate 2; the others were similar in principle, but no two were alike in detail.

Estimated cost.—The cost of the ring road, including property acquisition and works, would be about £123,000,000. In addition, the three new surface link roads would cost nearly £9,000,000. On the other hand, two improvements at Gardiner's Corner and Vauxhall Cross costing nearly £2,000,000 would be incorporated. The net cost of the whole project would thus be about £130,000,000.

Economic value.—A motorway type of ring road round central London would be of great benefit to traffic, but it is important to consider whether a project of this magnitude would be economically justifiable. Estimates of monetary savings due to street improvements cannot be precise because of the assumptions which have to be made, but they can usually be sufficiently realistic to give a good indication of value, especially when comparing alternative schemes. The economic aspect of the ring road has been considered in two ways, comparisons between the cost of comparable sets of improvements and the estimated direct value of the ring road to traffic.

Method I

The County of London Plan proposed as the framework of a modernized street system in the central area of London:—

(1) An inner normal type ring road.
(2) An outer motorway ring road.
(3) Two fast cross-routes.

Assuming that it would be practicable to construct the latter the total cost of these projects might be about £320,000,000. Such a framework can reasonably be compared with the following:—

(1) An inner motorway ring road.
(2) An outer normal type ring road.
(3) Improvement to higher standards of five main radials between these rings.

These projects would cost about £230,000,000 or £90,000,000 less. They would probably be of considerably greater value to traffic than the first.

An alternative approach on the same lines was to consider the cost of increasing the street space in the area enclosed by the ring road to accommodate the additional vehicle-miles per hour which would be possible in the inner area if the inner motorway ring was constructed. Excluding such congested but costly routes as Oxford Street and Regent Street, and making due allowance for streets not fully loaded, about 40 miles of main streets would have to be widened or improved at a cost of about £180,000,000 to £200,000,000. If average speeds were increased a further expenditure of £120,000,000 to £150,000,000 would have to be incurred, mainly at intersections. A total expenditure of at least £300,000,000 has thus to be envisaged, which would be more than double the cost of the ring road.

Method II

In attempting to assess the value to traffic of the ring road the lack of precision owing to assumptions and variables can be reduced by relating factors, wherever possible, to current money values. The potential error arising from the fact that the construction of the ring road would take a considerable time and that during this period traffic volumes would no doubt increase, can be eliminated by considering, first, the hypothetical case of the direct effect of the ring road with present traffic volumes, and, next, with the maximum traffic volume reasonably possible with the ring road and the existing street system.

Financial advantage would result from:—

(1) Reduction in accidents.
(2) Lower costs of highway maintenance, lighting, and policing.
(3) Savings due to elimination or postponement of other street improvements.
(4) Savings to traffic.

A possible adverse factor is the effect on rateable values.

Savings due to reductions in accidents.—Monetary values can be placed on accidents and if Professor Jones's pre-war assessments [10] are multiplied by three for current values it can be assumed that serious or fatal accidents cost £2,100, slight injury cases £105, and damage only cases £30. From a detailed count of all accidents in 6 months of 1947 in the Metropolitan Police District the intensity of accidents in the inner area was 7·8 times that in the area as a whole. In general terms, accidents vary proportionately with the rate of traffic flow. On this basis, it is considered that as the result of attracting traffic to the ring road there would be seventy-five serious or fatal, 500 slight injury, and 500 damage cases fewer in the inner area each year, representing an annual saving of £225,000. Against this must be offset the cost of accidents on the ring road. Such accidents should be few but, owing to the higher speeds, would tend to be more severe. If twenty serious or fatal, twenty slight injury, and forty damage-only cases (with the cost of the latter doubled) be assumed the annual cost would be £46,500 reducing the net saving to £178,500.

After the ring road has been completed and traffic volumes increased until a natural equilibrium has been reached on it and in the inner area, the accident position would probably be much the same as now in the inner area and on the ring road there would be rather more accidents, but with a reduction in severity. The net saving due to the ring road would thus be reduced by a percentage which can only be conjectural. It is considered that, because of the rather arbitrary nature of the assumptions and of the relatively small proportion of accident savings to total savings, the financial effect of accidents should be ignored.

Miscellaneous savings.—It is probable that the net cost of highway maintenance, lighting, cleansing, and policing would be reduced because of the ring road, but any assessment of its monetary value involves so many items and assumptions that it would be of little use. In any case the total savings would not be appreciable.

Savings in other street improvements.—The range of possible variations in any street-improvement programme is so wide that it would be difficult and arbitrary to attempt to place a value on them. All these improvements would have a traffic value whether the ring road was constructed or not. Except, therefore, for two cases on the alignment of the ring road which can be credited to the cost of its construction, any saving under this head is ignored.

Effect on rateable values.—If premises are demolished the income to the local authority concerned is reduced by the loss of the rates on those premises and the total loss of rateable value after construction of the ring road was estimated in 1948 to be about £350,000. On the other hand, the increased demand for premises in the vicinity, their consequent change of user, and the construction of new buildings, would largely offset this loss. Although some local authorities might suffer some reduction in income, it is doubtful whether rateable values as a whole would show any appreciable loss. Accordingly this factor, too, may be eliminated.

Savings to traffic

Savings to traffic include reduced wear and tear of vehicles, lower consumption of fuel and lubricants, and time. Considerable investigation of these items shows that all except the time factor should be ignored. The rate of wear of vehicles, especially tires, increases with frequent stops, starts, and changes of speed. On the other hand, higher speeds and distances increase the rate of wear. The mixed nature of the traffic involved, the dearth of sufficient reliable data, and the tendency of credits and debits to cancel each other make reliable estimates impossible. Much the same considerations apply to fuel and oil consumption. The net extra fuel consumption by vehicles using the ring road would be offset to a considerable extent by the savings accruing to traffic in the inner area as a result of the greater uniformity of speed. Any monetary estimate of the net extra cost of fuel and lubricants would be unrealistic and, because it would be small in proportion to other savings, has been ignored, except in the case of ring-road traffic. An interesting fact, although of relatively small economic importance, which emerged from this study was that a few very heavy commercial vehicles with high fuel consumption would find it advantageous to use a direct shorter but slower route than to save time by using the ring road. The remaining item, time, on the other hand, proved to be of vital importance and its examination must be more detailed.

The assumption was made that vehicles which would save time compared with existing average speeds by using the ring road should be used as the basis of any calculation. This introduces a relatively small error because a few vehicles would not save time in this way after the ring road had increased average speeds in the inner area, but this error would tend to diminish and would disappear when equilibrium was reached with increased traffic volumes. This factor is therefore ignored.

Savings due to economies in time can be divided into:—

(1) Those accruing to vehicles using the ring road.
(2) Those accruing to vehicles using the other streets in the inner area.

The first step is to place values per hour and per mile upon the various classes of vehicles. Some are more difficult to estimate than others and all can be assessed

on differing bases. Estimates or data[11, 12] of running cost and hire rates were made or obtained and all showed striking similarity. Average values are estimated to be:—

Class of vehicle	Value per hour: *s*	Running cost per mile: *d*
Light commercial vehicles	6·33	5·23
Heavy „ „	8·00	7·64
Cars, including passengers	21·50	4·54
Taxis, „ „	15·80	4·54
Motor cycles	5·75	2·12
Buses and coaches (excluding passengers)	16·25	10·79

Taking the ring-road traffic first, the proportions and average numbers of the main classes of vehicles are estimated to be:—

Class of vehicle	Percentage	Average number of vehicles per hour
Light commercial vehicles	30·26	1,483
Heavy „ „	6·31	309
Cars	44·39	2,175
Taxis	13·67	670
Motor cycles	3·97	194
Coaches	1·40	69
	100·00	4,900

All these vehicles would save time ranging from short periods to a maximum for through traffic travelling along half the ring road. Taking mean journey distances and equivalent straight-line speeds on existing streets the savings in time may be estimated as follows:—

Radial traffic		Min
Time taken on existing streets:—		
2·5 miles at 10 m.p.h.		15·00
Time taken using ring road:—		
On existing streets = 0·54 mile at 10 m.p.h.	3·24	
Entering and leaving ring road	1·00	
On ring road:—		
2·88 miles at 36 m.p.h.	4·80	
		9·04
Time saved:		5·96

Through traffic		
Time taken on existing streets:—		
3 miles at 10 m.p.h.		18·00
Time taken using ring road:—		
Entering and leaving	1·00	
On ring road:—		
3·75 miles at 36 m.p.h.	6·25	
		7·25
Time saved:		10·75

Through traffic may be taken as being 15% of the whole, and integrating the time saved in respect of both groups of traffic, the average time saved per vehicle was 6·68 min. Taking the number of vehicles earlier in this paragraph, their mean time value per hour as 14·6*s*, a working day of 12 hours, and a year as 300 working days (to allow for week-ends and Bank Holidays) the annual saving would be:—

$$\frac{6{\cdot}68}{60} \text{ hour} \times 4{,}900 \text{ vehicles per hour} \times £\frac{14{\cdot}6}{20} \text{ per vehicle hour}$$

$$\times 12 \text{ hours} \times 300 \text{ days} = £1{,}434{,}000$$

This figure requires adjustment because of the cost of the additional mileage run, offset by the saving in fuel consumption per mile. The average journey distance on the ring road is 2·66 miles. The extra distance for radial traffic is

$$(2{\cdot}88 + 0{\cdot}54) - 2{\cdot}5 = 0{\cdot}92 \text{ mile}$$

and for through traffic 0·75 mile; integrating this as before, the average extra mileage per vehicle is 0·89. If a fuel economy of 20% is assumed the extra distance on the ring road may be regarded as reduced by 10% and integrating the average group mileages the equivalent extra mileage becomes $0{\cdot}89 - (0{\cdot}1 \times 2{\cdot}66) = 0{\cdot}63$ mile. The average running cost per mile of the ring traffic would be 4·94*d*. The annual cost would be:—

$$0{\cdot}63 \text{ mile} \times 4{,}900 \text{ vehicles/hour} \times £\frac{4{\cdot}94}{240} \times 12 \times 300 = \text{say, } £229{,}000$$

The net annual saving in respect of ring-road traffic would thus be, say, £1,205,000 on a conservative basis.

Secondly, an attempt has been made to assess the financial effect of the ring road on traffic *not* using it but, owing to the increased number of variables, this presents more difficulty. Possibly the best approach to this problem is to consider the value of the saving in time to traffic in the inner area owing to its increased average speed arising from the relief afforded by the ring road. The total traffic volume on the principal traffic routes in the inner area may be taken as 102,000 vehicle-miles per average hour. The ring road may be expected to reduce this volume to, say, 70,000 vehicle-miles/hour. This reduced volume could run at a higher average speed with a consequent saving of journey times. The vehicles would not be of special types as in the case of the ring road and, because of the higher proportion of commercial and public service vehicles, the value of the time saved would be greater than that of the ring-road traffic. The proportion of time saved and the value of the time of bus passengers are the most important factors involved in estimating this value.

The amount of time saved would depend upon the increased average speed on the principal traffic routes in the inner area. The present average actual speed may be taken as 12 m.p.h. for all vehicles and, from a considerable amount of comparable data, the average speed after the relief from the ring road would be at least 14 m.p.h., or an increase of 16·7%. The time saved would be 14·3%.

Some economists consider that the value of the time of passengers in public service vehicles should be ignored. If, however, 2*s* per hour per passenger and an average load of thirty passengers per vehicle were taken, the financial saving to traffic as a whole would be increased by about 90% whether or not cycles are included.

It is next necessary to estimate the value of a vehicle-hour for the traffic left in the inner area. The proportions between the different groups of traffic in 1954 and the proportions after the removal of the ring-road traffic are given on the following page.

Class of vehicle (1)	Percentage based on fifty-four points in 1954 police count (2)	Percentage after deduction of ring-road traffic (3)
Light commercial	24·3	19·0
Heavy „	4·3	2·7
Cars	34·7	26·3
Taxis	13·2	15·7
Public service vehicles including coaches	11·9	19·6
Motor cycles	3·1	2·3
Motor-cycle combinations and motor-assisted cycles	1·6	2·7
Pedal cycles	6·6	11·2
Locomotives, tractors, horse-drawn vehicles, and barrows	0·3	0·5
	100·0	100·0

Applying the values per vehicle-hour on p. 119 to the percentages in column (3) above, but ignoring cycles, non-mechanical vehicles, and barrows, the value of an average vehicle-hour is 14·7*s*. But the classes of vehicles whose time value is ignored occupy road space and including them from the volume aspect, the value of a vehicle-hour is reduced to 13·2*s*.

After the ring road has relieved the inner area, the number of vehicle-miles per hour in it becomes 70,000, equivalent, at 12 m.p.h. to about 6,000 vehicles. If the average speed rises as little as 2 m.p.h. and thus causes a saving of time of 14·3% the total annual saving would be:—

$$14{\cdot}3\% \text{ of } 6{,}000 \times £\frac{13{\cdot}2}{20} \times 12 \times 300 = \text{say, } £2{,}038{,}000$$

If bus passengers' time were included the saving would be £3,872,000. The higher the average speed after the relief afforded by the ring road the greater would be the saving; if it was 15 m.p.h. the saving (exclusive of passenger time) would be £2,850,000 and if 16 m.p.h. more than £3,500,000.

Taking the savings to traffic on the ring road and in the inner area together the total, excluding bus passengers' time, becomes £3,250,000 per annum with present traffic volumes.

It is, however, highly probable that traffic volumes generally will increase and one of the principal advantages of the addition of the ring road to the existing street system is that a considerable growth of traffic volume could be accommodated before conditions in the inner area again approximate to those obtaining at present. It has been shown that both on the ring road and in the inner area an increase of about 50% in traffic volume could be accommodated. The value of the time saved by the increased ring-road traffic at a lower average speed, would be nearly £2,000,000. The value of being able to accommodate 50% more traffic in the inner area is, however, not capable of quite such straightforward determination, because average speeds would be either about the same or slightly less than at present.

The increased volume in the inner area will be 102,000 — 70,000 = 32,000 vehicle-miles/hour and it is necessary to estimate the value of running a vehicle-mile. This figure, or some of its elements, can be used as a basis for estimating the gain to the community. First, most vehicles in inner London during the working day are not run for pleasure but for profit and, if a flat rate of 25% is assumed, the gain to an owner from each vehicle-mile is thus 4·37*d*. Secondly, the community gains from

taxation revenue. Under "journey" cost road-fund tax averages £30 per vehicle and at, say, 8,000 miles per year is equivalent to 0·9*d* per mile, and petrol and oil taxes account for a further 1·43*d* per mile. The total gain per vehicle-mile is thus:—

$$32{,}000 \times £\frac{6{\cdot}7}{240} \times 12 \times 300 = \text{say, } £3{,}200{,}000$$

Adding this to the additional savings in respect of the increased ring-road traffic the total annual saving at present prices after the 50% increase in traffic volume would be £5,200,000. Because the ring road would require several years to construct this figure is more realistic than that given on p. 121.

The cost of the ring road at present prices would be about £130,000,000. If it is assumed that property acquisition and construction would take 10 years, that the capital cost would be met cumulatively and proportionately, that a 60-year loan-repayment period is reasonable, and that the interest rate would be 3½%, the annual loan charges from the 11th to the 50th years would be about £5.200,000, precisely the same as the value to the community of the ring road.

Other planning considerations

A major engineering project can seldom be considered from the aspects of cost and construction alone. Limitation of space precludes more than brief references to other salient considerations.

Amenity.—On the ring road itself low retaining walls with grassed banks planted with small flowering trees and shrubs would provide a pleasing and restful appearance, not only to road users, but also from bridges and buildings. The latter would be sufficiently distant to reduce traffic noise to a level considerably below that in an ordinary street. As many bridges as necessary to maintain the existing street pattern could be constructed and the ring would be easier for pedestrians to cross than most main roads on the surface. Property demolition would probably pose fewer problems in treating exposed but unsightly elevations of buildings than a normal street improvement. Objections are frequently made to viaducts, but these are probably due in the main to the unaesthetic activities beneath the arches of railway viaducts. Little criticism is heard of the appearance of Waterloo Bridge where it spans Victoria Embankment, and there is no reason why suitable architectural treatment of the ring road, where in viaduct, should not produce structures of pleasing and graceful appearance.

Rehousing.—Rather more than 19,000 persons would be displaced, and all except 2,000 would need to be offered alternative housing accommodation. To satisfy this more than 200 acres would be required if flats were built, or 625 acres for houses. In 1948 there were considerable arrears of normal housing still to be overcome, but the present position is such that this additional commitment could probably be accepted without undue difficulty. The reinstatement of office and business accommodation would be more serious. In this connexion roofing of parts of the sunken portion of the ring was investigated, but only on one short section would the value of the area thus regained have exceeded the structural cost. The artificial lighting necessary would reduce both the attraction of the road for drivers and its capacity, and the legal complications would be extensive. On balance, building over the ring is considered generally inadvisable.

Powers.—Adequate powers appear to be contained in the following statutes:—

(1) The Town and Country Planning Act, 1947, in which Section 47 incorporates the powers of Section 10 of the Development and Roads Improvement Funds Act, 1909.

(2) The Metropolis Management Act, 1855 and the amending Act of 1865.
(3) The Special Roads Act, 1949.

The last named was amended in its Bill stage to enable roads of the "A" ring type to be built, and for the nature of its traffic, facilities for the apparatus of statutory undertakers and other matters to be strictly controlled.

Conclusions

The fact that the loan charges of the ring road are equivalent to its value is not necessarily the final verdict on the proposal. The assumptions made in putting values to vehicle time, speeds, and other factors are conservative and minor changes would result in a substantial gain. On the other hand, the interest rate taken is lower than at present. The main point seems to be that, if a 50% increase of traffic volume is to be accommodated in the central area of London, the ring-road project would enable this to be achieved at less than half the cost of a comparable set of normal surface widenings and intersection improvements. There are the further assets of the prestige value of a single imaginative project and of its tactical value in war. On any reasonable assessment of the cost and of the direct and indirect benefits the Author considers that the "A" ring motorway would be both practicable and justifiable.

Acknowledgements

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The opinions expressed in the Paper are those of the Author and do not necessarily reflect the views either of Government Departments concerned or of the London County Council or of the chief officers of these authorities.

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The Paper, which was received on 31 August, 1955, is accompanied by fourteen sheets of diagrams from which the Figures in the text and folding Plates 1 and 2 have been prepared.

Discussion

The Chairman remarked that in nearly all plans of towns which had been made the planners had included a ring road, because it had been fashionable to do so, but the Chairman believed that in many smaller towns a ring road would be more trouble than it was worth, because it would take traffic away from what corresponded roughly to a straight line and make it go in a circle. The problem in relation to London was peculiar to London. He had no doubt that the discussion would clarify their ideas on whether or not a ring road was wanted. The Paper was a highly scientific analysis of the problem, and the fact that practising engineers could deal with a matter of that kind on a scientific basis was a great testimony to engineers in general and to the Author in particular.

Dr W. H. Glanville (Director of Road Research, D.S.I.R.) said that the Paper was very comprehensive and gave a great deal of interesting historical information. The Author indicated that most American cities, apart from Washington, had a rectangular street pattern and that ring roads were inapplicable, but Dr Glanville wondered how true that was. He felt from his own experience in America that the Americans were tending to construct, and had in fact constructed, a number of ring roads in different cities and that those which had been constructed had been found to be extremely valuable. The American practice was to provide roads right through the centre of the city and the urban area which could be looked on as radiating roads in one sense, and in some cases to provide inner ring roads as well. All those were "free" ways, i.e., motorways with intersections at different levels from the roads which they crossed. It seemed that that was a principle which must be accepted in future for all large-town development if the full advantages offered by motor transport were to be obtained, because the time saved and the convenience of travel were much greater than with intersections at grade. In England there had been almost no experience of that, and so the public did not realize the great advantages given by intersections which were free from traffic flowing across them. He hoped that the Author's obvious desire to see a ring road built on such principles would be fulfilled.

The Author had indicated some of the obstacles in the way, not the least of which seemed to be that the planners had felt that proposals involving such expenditure would not be acceptable to the Government. That attitude had important repercussions which it was necessary to consider. Moreover, the historical part of the Paper seemed to show that some of the planners had regarded traffic needs as a secondary matter. That was an extraordinary attitude to adopt, and any engineer would feel, with the Author, that such an approach lacked realism.

Another extraordinary fact brought out in the Paper was the foresight shown by the builder of the Crystal Palace, Joseph Paxton, in his proposal for a covered arcade. It would have been a great blessing if some arcades of that kind had been constructed in the nineteenth century.

FIG. 6.—GROSVENOR ROAD DESTINATION COUNT (29TH APRIL, 1948)

FIG. 7

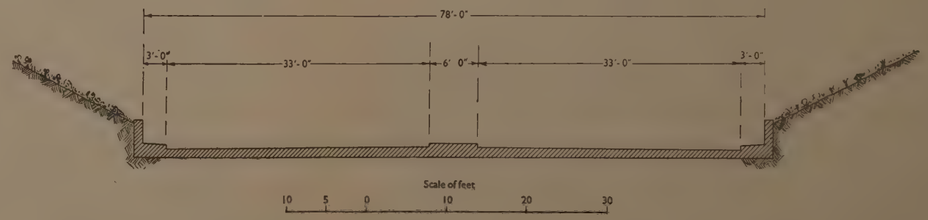

FIG. 10a.—TYPICAL CROSS-SECTION IN OPEN CUT OF SUNKEN SECTIONS

WILLIAM CLOWES & SONS, LIMITED: LONDON

THE PLANNING OF RING ROADS WITH SPECIAL REFERENCE TO LONDON

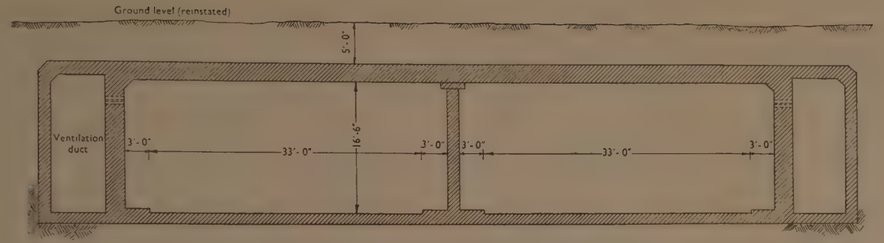

FIG. 10b.—TYPICAL CROSS-SECTION OF UNDERGROUND SECTION IN CUT-AND-COVER

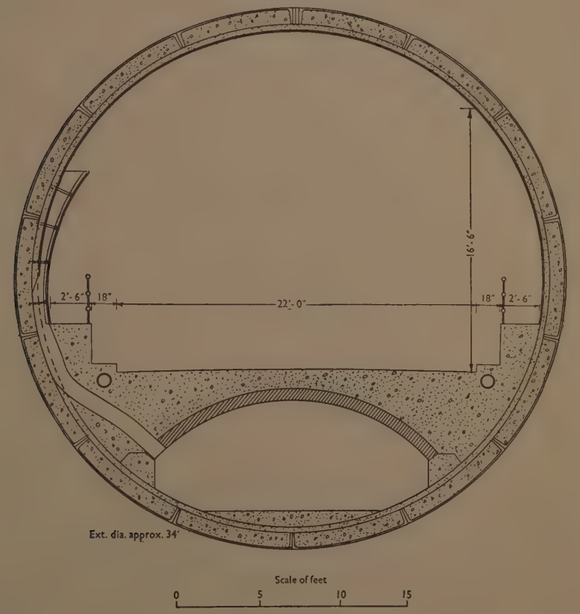

FIG. 10c.—TYPICAL CROSS-SECTION OF SUB-AQUEOUS TUNNEL SECTION (ONE CARRIAGEWAY ONLY SHOWN)

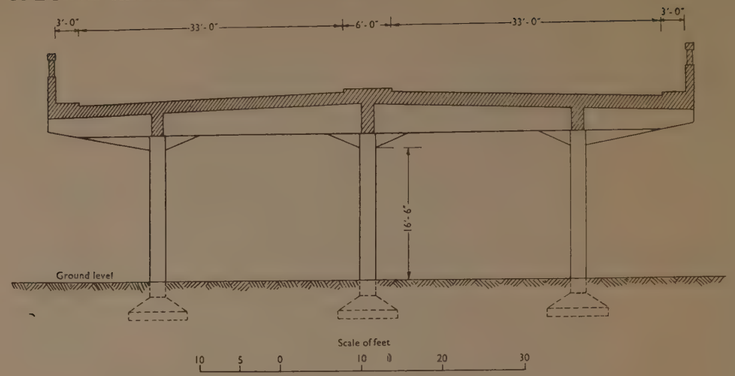

FIG. 10d.—TYPICAL CROSS-SECTION OF SECTIONS OF VIADUCT

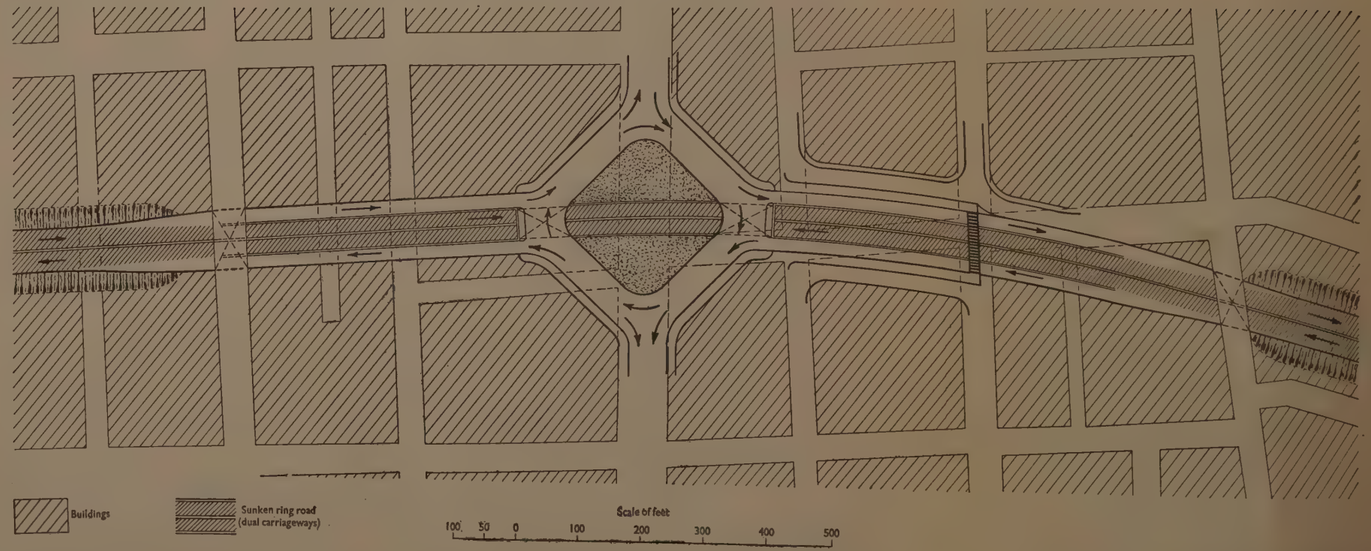

FIG. 11.—LAYOUT OF TYPICAL INSECTION AND ACCESS POINT

If the present London development plan were carried out the advantages, as the plan stood at present, would be of a minor character. It had been estimated by some of Dr Glanville's colleagues at the Road Research Laboratory that if all the improvements envisaged were carried out the effect would be to increase the speed of vehicles by only about 1 m.p.h. on the main routes as compared with the speed of traffic at the present time. With roads which were free from intersections, on the other hand, the improvement in speed would be enormous, and the 35 m.p.h. which the Author mentioned as the probable average speed on such a road seemed to Dr Glanville to be very low in relation to the experience of other countries; he would expect, if such a road was properly planned and built, that average speeds in excess of that figure would be possible, particularly when the modern high-speed motor-car had more completely ousted the older and slower cars, as must happen in the course of the next few years. Such higher speeds would have a corresponding effect on the economies due to savings in traffic time.

Throughout the calculations and arguments which the Author put forward he spoke of the dearth of reliable data. Under the heading "Savings to traffic" on p. 118 he said that the effect of that was to "make reliable estimates impossible," and in the next paragraph he wrote "The assumption was made . . ." Earlier he referred to certain "provisional assumptions," which led to rounded-off figures of percentages. It seemed to Dr Glanville that that was a very important point, because the Author had presented an up-to-date Paper as a result of all the information at his command in the London County Council, and yet it was based upon scrappy and incomplete data and information. One of the most important lessons of the Paper was how much it was still necessary to know, how much information had still to be collected, and how essential it was to collect that information as soon as possible.

To some extent the collecting of information was going on, and the Road Research Laboratory was helping towards it by making the traffic surveys and so on to which the Author had referred, but a great deal more was required. For example, the Americans carried out very elaborate origin and destination studies by interviews in homes to try to assess on a statistical basis where the traffic came from and where it wanted to go, and even in such a large town as Chicago, which was comparable in size to London, that task was undertaken. As a result, of course, they had a great many figures to analyse, and in the past that had taken up to a year for a big town; but with modern electronic techniques the analysing and sorting out of the data would be done much more quickly than in the past. He had been told that in Detroit the first survey had taken about 11 months to work out in order to obtain the necessary information. A second survey, for a town of about the same size, using a simple electronic machine, had taken about 3 months, and a third they expected to take a matter of days, or perhaps even hours. How true that was he did not know, but it indicated that there were possibilities, in tackling the problem of a huge town, which had not existed before.

It was important when any work of that kind was done in the future and when plans were drawn up that much more time should be devoted and care given to obtaining data. A good deal of research would probably be required into the methods of obtaining data, but the cost, although it might appear high, was not high in relation to the cost of the schemes themselves; it would probably be not more than 1% of the cost of the scheme. Planning by "hunch," as had been the case for some of the plans mentioned in the Paper, should be a thing of the past.

The problem was, of course, very complex indeed, because not only road traffic but also other forms of transport should be considered. It had been realized in the United States of America that it was no good planning for roads alone; it was necessary to take into account all other forms of transport and deal with the problem as a whole. That would have to be done in Britain. There was also the problem of parking, but he noticed that the Author had kept clear of that, and he had better do so too.

Mr H. S. Andrew (County Engineer and Surveyor of Middlesex) said the Author seemed to have grasped the main reason in favour of the "A" ring which would appeal to the practising engineer when he concluded that "if a 50% increase of traffic volume is

to be accommodated in the central area of London, the ring-road project would enable this to be achieved at less than half the cost of a comparable set of normal surface widenings and intersection improvements." That was one of the strongest arguments in favour of the "A" ring which could be produced.

In Part I of the Paper, under the heading "Official views," the Author quoted the Built-up Areas Report, quite properly, as recommending a single ring road for a town of 20,000 people and three or more for towns with a population of over 100,000. The opening remarks of the Chairman were apposite in that connexion. In the case of an urban area as big as that of Greater London, Mr Andrew wondered why any doubt had been expressed about the need for the five ring roads proposed in the Greater London Plan; surely a population of 8,000,000 required those roads?

Mr Andrew agreed strongly with the Author's conclusions at the end of Part I of the Paper, except that he had some doubt about the latter part of (3) as applied to Greater London. He referred to the statement that internal local traffic would tend to decrease as the area enclosed by the ring increased. He would be prepared to agree with that as a generalization, but was inclined to think that Greater London must be regarded as a collection of separate towns, or at least of separate industrial areas, and that those ring roads would in effect serve the purpose of radials to those areas alongside which they ran. The traffic on them would largely be short-distance traffic, and he agreed with Dr Glanville that the methods of traffic census for dealing with that vast problem required looking into. The method which came most readily to mind for Greater London was the sending-out of a questionnaire to the owners of all vehicles normally garaged within a certain radius of Greater London. He appreciated that that would be a vast undertaking but in the end it would prove to be a paying proposition.

On p. 104 the Author also stated that ring roads should normally be constructed on the motorway principle, "specially designed for high speeds, with a limited number of carefully designed intersections . . ." and so on. Mr Andrew did not think that any engineer concerned with roads would suggest the building of a ring road on any other principle. He was puzzled, however, to find that on p. 116, under the heading "Method I," the Author was considering two alternatives, in each case under (1), (2), and (3), and that under each of the alternatives he appeared to suggest that either the inner or the outer ring should be a road of normal type. It was difficult to understand why. The Author might have some reason for suggesting that the "B" ring road should be a normal type of road in the second series, and it would be interesting to know whether his figure of £230,000,000 provided for building the "B" ring as a normal type road. If that were so Mr Andrew was doubtful about it, being of opinion that it would be cheaper to build it of the motorway type with the restricted width of 78 ft as compared with building it to accommodate the general access and the standing traffic which would be generated on a road of that type in those circumstances. Perhaps the Author could give some figures on that.

The next point was perhaps trivial, but the Author referred twice to the objection to viaducts. Engineers were suffering at the present time from a strong public objection to viaducts, due not to the viaduct itself, which could be a very beautiful structure, but to what the Author wisely referred to as the unaesthetic activities which went on underneath the arches. The objections which were always raised by the general public and by the planners when there was any talk of putting a road on a viaduct were very largely founded on the sort of thing which had gone on in Central London and in other large cities, where the desire had been to get the utmost use out of every square foot of land.

Mr A. J. H. Clayton (Divisional Engineer, Parliamentary, Improvements, and Town Planning Division, Chief Engineer's Dept, London County Council) said he wished to emphasize some of the points made in the Paper. There was, for example, the question of "through" traffic, on which the Author had commented. It was important to remember that it was not possible to talk about through traffic or local traffic, or even about the mysterious "local-through" traffic of the planners, without relating it to an area. In respect of Bond Street, the traffic which went from one end to the other was through traffic, but it was local traffic, or might be, in respect of Inner London. Traffic from Birmingham

to Dover was through traffic in respect of London, but might conceivably be called local in respect of Europe as a whole. To be logical, traffic could be described as "through," "local," or "radial" only in respect of some area.

One point which the investigation described in the Paper had brought out was the importance of radial traffic—"radial" in respect to the area enclosed by a ring road—in considering the use which was made of the ring road. The popular idea in the past of by-passes, ring roads, etc., was that they were to divert through traffic from urban or residential areas as if through traffic was potentially more dangerous than local traffic. Probably one of the most dangerous vehicles, in proportion to its mileage, was the electric milk-delivery van, at any rate so far as young children were concerned, and that could hardly be called a through vehicle. The importance of radial traffic was brought out on p. 112. At least half the relief to the inner area, according to the Author's calculations, came from the radial traffic being taken away, and 60% of the traffic on the ring was radial traffic, not through traffic.

On p. 108, the increase in traffic between 1937 and 1954 was given as 22·4%. That might give rise to some misunderstanding; it should be emphasized that that related to twenty points on the fringe of the inner area. It should not be supposed that percentage was general for London as a whole. It might arise from people who had cars on the firm's account driving them to London and parking them there. Neglecting bicycles (as did the figure of 22·4%) the increase since before the war at 100 points in Greater London as a whole, most of them towards the centre but quite a few of them outside, was 15%. The number of bicycles had decreased, which helped a good deal, and most of the increase came from light commercial vehicles and private cars, which did not take up so much room as lorries and buses. That was the increase in actual observed traffic; the increase in vehicles licensed was much greater. The difference was partly due to the inhibiting effect of congestion on London's roads and partly to the fact that London was growing outwards and people used their vehicles more for taking journeys out of London than for going into London.

On p. 114 reference was made to the effect of speed of vehicles on the ring road on its capacity as a traffic-carrying unit, and in that connexion Dr Glanville had expressed the view that a higher average speed than that foreseen by the Author might be possible. It might be in the future, but for purposes of present calculation higher average speeds should not be assumed for a road open to all classes of traffic. There were many lorries which travelled at more than 50 m.p.h., but there were also many people who wanted to go more slowly, or perhaps were unable to go fast, and with large volumes of traffic those slower vehicles would reduce the average speed. With a motorway of the kind projected many drivers would like to go at 60 to 70 m.p.h. if given the opportunity to do so, but when calculating use it was necessary to remember that large volumes meant a reduction in speed.

In a recent discussion Dr Charlesworth[13] had given some data for a 30-ft dual carriageway—not a motorway, but virtually so for the purposes of the present investigation—and showed that up to 500 vehicles/hour/lane the speed was constant at 35 m.p.h. It had not been possible to observe above 500 vehicles/hour/lane because there had not been more. On a rather narrower road, half a dual-carriageway with a width of 25 ft, the average speed started to drop when a figure of about 500 vehicles/hour/lane was reached. It was necessary, therefore, to allow for the fact that with large volumes of traffic and all classes of traffic permitted to use the road there would be some limitation on the average speed which could be used in making calculations. It might be higher than that quoted by the Author, but there would be a limitation and that must be taken into account. Moreover, it was relative speed between the ring-road motorway and the local roads, which had a particular effect when dealing with traffic such as radial traffic, which had to go a good deal farther to use the motorway.

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[13] A. J. H. Clayton, "Working Capacity of Roads," Proc. Instn Civ. Engrs, Pt. II, vol. 4, p. 652 (Oct. 1955). *Disc.* by Charlesworth, p. 689.

The Author deliberately dealt with a ring road, but there were other possible methods of dealing with the problem. In the case of a small town two cross motorways might be provided instead of a ring, but that would provide for only four radial roads.

If, as was the minimum for London, there were to be eleven or twelve radial motor roads instead of a ring a large mileage of roads would have to be built, and a difficult situation would be created where they joined in the centre of the town.

There might be other plans, such as that suggested by Mr Hounsfield, which was a sort of "noughts and crosses" grid pattern. To relieve a congested inner area something in the nature of a ring (though not necessarily a circle) while not necessarily the most effective method was probably the cheapest, and when talking in terms of a hundred million pounds some regard must be had to the availability of resources.

Dr G. Charlesworth (Head of Traffic Section, Road Research Laboratory) referring to radial roads, said that on p. 101 the Author gave some information about the proportion of traffic in towns of various sizes which could be by-passed. There was some American information which showed that, whatever the size of the town, about 20–30% of approaching traffic wished to go into the centre of the town. That suggested that the roads leading right through and into the centre of the town were of considerable importance, whatever the size of the town. Mr Clayton had indicated that those radial roads were important, and in Fig. 9 that point was emphasized, because it would be seen that there was more traffic on the radial road than on the ring. It would have been useful if the Author had elaborated a little on that. Inside the ring there were about 4,000 vehicles/hour in each direction, and they had to be distributed over the internal street system. Mr Clayton had mentioned the chaos which would be caused at the centre if all the traffic went through the middle of the town, and it would have been useful to hear a little more about how that vast volume of traffic could be distributed on to the existing street network.

On the question of intersection design, Dr Charlesworth emphasized a point made by Dr Glanville. Fig. 9 showed a roundabout system, and even with the 1954 volume of traffic there was too much traffic on the weaving sections for conditions to be satisfactory; he thought that they would be productive of discomfort and accidents, and he would not like to see a 45% increase in traffic on those weaving sections. It seemed that some alternative design was called for in a situation of that kind.

On p. 108 the Author made an estimate of the total vehicle mileage in 1954 on the streets in the central area. The Road Research Laboratory also had an estimate, of which the Author might not be aware, of the total vehicle mileage on all streets. The Author had been concerned with the major streets. At the Laboratory they had made an estimate of 127,000 vehicle-miles/hour for all streets during the same periods of the day, which was appreciably higher than the figure given by the Author. Quite a large proportion of those vehicles might use the ring roads, which would of course help to prove his case.

On p. 109 the Author said that 10 m.p.h. had been taken as the straight-line speed for the inner area. What speed had been used in the calculations given on p. 110, for the ring road? Was it the 30 or 36 m.p.h. referred to on p. 114?

Dr Charlesworth was a little doubtful about the calculation given on p. 112 of the radial traffic using the ring road. The Author took 21% of the approaching traffic, but it was not clear why he did not take 33%; 33% was the 41% of radial traffic using the ring deduced on pp. 110 and 111, multiplied by 0·8 (the proportion of approaching traffic which was assumed to be radial). If correct that would mean even more traffic on the ring road. On p. 113 the possible use of the ring road by local internal traffic was mentioned, but the Author did not appear to have made any estimate of how much of that traffic would use the ring road. That again, however, would mean some additional traffic.

Some speeds which Dr Charlesworth had quoted between intersections on the Great West Road had been referred to by Mr Clayton. If Dr Charlesworth remembered aright, those figures had been obtained in 1950 and speeds had been increasing at about 1 m.p.h. per year since then, so that they were now substantially higher, as Dr Glanville had suggested.

On p. 119 some values per hour were given for vehicles of different types, together with the running cost per mile. How had the Author deduced the figure of 21·50*s* for the value per hour of a car, and did that figure include tax? He might be interested in some preliminary results which had been obtained at the Road Research Laboratory for journeys in the Greater London area. They had found in 1954 for journeys made on week-days between 8 a.m. and 6 p.m. that the percentage of occupants travelling in working time was 5 for buses, 22 for taxis, and 29 for cars, the overall average being about 12%. That made a difference to the calculations, because working time could be costed in a way that economists would accept. They were hoping to have some information about the value of the occupants' time. For the occupants of cars it might be expected to be appreciably higher than the average, because the more highly paid people travelled by car, and the figure for them might well be more than 1½ times the average.

Mr C. I. P. Andren (Lecturer in the Department of Civil and Municipal Engineering, University College, London) said that the Author, in referring to the road proposals of the consultants' County of London Plan, remarked that "Traffic needs seem to have been secondary," and that point had been taken up by Dr Glanville. Mr Andren believed it was unfortunately true that that attitude was still extremely common, and, although he was delighted to see the Author's factual approach to the ring proposal, and his advocacy of a relief road with multi-level junctions, there was a danger that the Author was, perhaps unwittingly, encouraging planners and others to go on thinking in terms of putting a uniform-width ring road on the map of London, regardless of traffic needs and without first obtaining essential traffic information from origin and destination studies and then planning the additional road space in those areas where it was most needed. The Author and his colleagues were, of course, far too experienced to wish to impose a ring as such on London without regard to traffic and topography, but Mr Andren was afraid that such an attitude was still extraordinarily common.

In the past few years there had been many headlines in local newspapers such as "Councillors discuss where to put the ring road." There might possibly be a case for thinking in terms of a geometrical pattern, whether a ring or a grid, in a completely featureless plain which was empty of development at the present time and which was to have a city built upon it, but the Author seemed to make it clear on p. 101 that even in a case of that kind the ring was probably the least effective type of geometrical pattern.

It would be just as logical to plan a road system on such a plain on the principles of Vitruvius. Vitruvius's ideal town had a circular outline and the houses were to be arranged in radial lines from the centre, in such a way as to avoid the eight poisonous winds which he maintained blew from certain points of the compass. It would seem that Vitruvius was in favour of ring roads, and perhaps the supporters of ring roads could draw some comfort from his opinion; he had been, after all, consulting engineer to the Emperor Augustus.

It had been suggested that if a group of educated people were asked to design the layout of a road to carry the least possible traffic, they would eventually come to the conclusion that the road should be sited at right-angles to the major flows. It was known that the major flows between towns and between important centres within big cities were mostly radial, and the conclusion from that, presumably, was that the only layout which would give a continuous road at right-angles to the radials would be a ring, so that it was a ring road which would carry the least possible traffic.

In Part III of the Paper the Author dealt with the case for a ring road in London, and in his second paragraph quoted some week-day average flows. He referred to an average of 18,000 vehicles/hour, excluding bicycles, entering the inner area. From Plate 1 it would appear that the Author's inner area corresponded very closely to Hounsfield's cordon. The figures given there agreed very closely with the Author's figure of 18,000. Hounsfield's figures were for 1937.

At the top of p. 109 the Author came to the conclusion that only about 15% of the vehicles circulating in the inner area was through traffic. That seemed to imply that of the 18,000 vehicles/hour in 1937 only about 2,700 would be likely to be approaching the

Author's ring tangentially and would therefore have wanted to follow at least a part of the ring, while the other 15,300 would have tended to cross the ring at right-angles. If that were correct, Mr Andren thought that there must be some doubt about whether the ring was the most suitable form from the functional point of view; it seemed to be more logical to construct short lengths of relief road in the general direction of the main traffic movements.

On p. 116 the Author said it had been eventually decided that there should be nine access points. If the ring was going to carry the flows intended the concentration of traffic at those points, both inside and outside the ring itself, was going to be extraordinarily difficult to handle. That seemed to be borne out, as Dr Charlesworth had mentioned, by Fig. 9. The Author might be thinking in terms of perhaps 500 vehicles/hour/lane for the approach roads of the normal street system at junctions of that kind; if so it would appear that flows of 4,000 vehicles/hour in each direction would need eight traffic lanes each.

If the Author's ring was not intended to be the end of all major development of the London road system, what further desirable improvements would he suggest? Would he go on to include radial roads, perhaps building radials of the grade separation type? If so, and if the fact was to be accepted that it would be necessary in future to go beyond even such a project as the ring motorway, might not it be better to start from the beginning with a really long-term plan, perhaps on the lines of Hounsfield's original layout of a series of relief roads for London?

Mr Andren showed a slide of the first two stages which Hounsfield had suggested for the relief roads. Those were to be of the same character as the Author's ring road, with junctions on at least two levels. Hounsfield had wanted to start in Hyde Park, because that would not involve the problem of the demolition of property, but as time went on his roads would involve an enormous cost in demolition and construction. By spreading the work over a long period, however, and working to an overall plan, he had hoped to provide a means of coping effectively with London's traffic problems.

Mr W. T. F. Austin (Civil Engineer, Freeman, Fox and Partners) suggested that the Author had, in his approach to the subject, put his finger on what was the most important problem. The biggest obstacle in the way of solving London's traffic congestion was lack of money. Any scheme for solving the problem must be in terms of motorways, and any such scheme in central London or any comparable area was bound to be expensive. It was difficult to get the people who controlled the spending of money in Britain—the politicians—to appreciate that that money must be spent. The Author rightly approached the problem from the point of view of convincing them that, if it was not spent, an even larger sum would be wasted. The Author showed that the ring road which he described would save each year an amount of money equivalent to the annual charges on a 60-year loan.

Although the principle was right, Mr Austin thought that the result was not very encouraging. There were many projects in Britain at the present time, requiring large sums of money, which would also save large sums; many of them would save far more money, as a percentage of capital expenditure, than the one in question.

Mr Andren had made the main point which Mr Austin had intended to make, namely, that most of the traffic was in a radial direction. The Author had realized that, and had therefore decided that the roads carrying that traffic must be relieved. That seemed to be the object of the ring road, and the Author finished up with motorways on the ring road in a direction at right-angles to the roads on which he wanted to relieve the traffic, leaving them as ordinary surface roads. That was illogical and, although he had attacked the problem in the right way, he did not seem to have found the right answer. The Author had pointed out that the Paper dealt with an investigation of a ring road and did not claim that the ring road was the only or necessarily the right solution. It would be interesting to know if he had investigated in the same way the provision of motorways in an outward direction—not necessarily exactly radial, because that would lead to difficulties at the centre, but generally outward, spreading the load from the acess points by means of surface roads.

Traffic engineering was not an exact science, and the assessment of the savings arising from increased speed of traffic was again by no means an exact science, but very largely a matter of opinion. Mr Austin thought that all future schemes should be analysed in the same manner to see whether the cost would be covered by the savings which would result.

The Author had looked into the economics of traffic congestion, and it would be interesting to know whether he had any idea of what the total cost of congestion in London was. Some attempt should be made by a responsible authority to discover that and give it great publicity, so that the money required to solve the traffic problem would in due course be made available. From the Author's own figures the cost of congestion in the central area of London, inside his ring road, would be £10,000,000 per year, and for the whole of London it might be about £50,000,000.

Mr Austin referred to the proposal to have in the ring a two-lane tunnel underneath the Thames near Tower Bridge, because a three-lane tunnel of circular cross-section would involve going deeper, with more difficult approach gradients. He asked whether consideration had been given to a tunnel of the caisson type, sinking a caisson through the bed of the river. That had been done several times and could be made as wide as might be desired without increasing the depth.

Dr Charlesworth had questioned the Author's figure of 21·5*s*/hour as the value for cars. Mr Austin, in the course of his work had had to make estimates of a rather similar kind, mostly in connexion with bridges, and had never used a figure of 21*s*/hour as the time value of a car, but something very much less. He would be grateful if the Author would say where that information came from.

The lane capacities quoted by the Author appeared to be reasonable for a uniform flow of traffic, but the following points should be remembered. Where hard shoulders were not provided, one broken-down car closed the equivalent of 1½ lanes (owing to the resulting congestion in the next lane). Experience on San Francisco Bridge had shown that during peak-hour traffic one vehicle per thousand broke down.[14] The near-side lane generally carried only about 75% of the traffic carried by the other lanes, even when all traffic was private cars. Lane capacities for commercial vehicles might be as little as one-third of those for private cars. The average spacing of junctions on the proposed ring road was approximately 1¼ mile and the nature of ring-road traffic was such that the journeys on the ring road would generally be short. There would therefore be much weaving between lanes which greatly reduced their capacity and increased the likelihood of bad accidents if the lanes were heavily loaded. For that reason during peak hours the 45% reserve capacity mentioned by the Author was unlikely to be found in practice.

Mr S. Kerr-Nesbitt (Assistant Valuer, London County Council) said that after the 1939–45 war the London County Council had been thinking in terms of property and engineering works of the order of £10,000,000 per annum. That, however, had soon been shown to be unrealistic and at the time of the investigation of the ring road had been reduced to £6,000,000 per annum. Not long after that, cuts had been made in capital expenditure and for the past few years they had been spending only £200,000 per annum. Quite recently the cuts in capital expenditure had been partly restored, to a level of about £2,000,000 per annum.

When such an annual sum was compared with the figures which had been given that evening, i.e., £130,000,000 for the ring road, it would obviously be a long time before the new road was built. Mr Kerr-Nesbitt thought that road engineers should give some of their basic proposals for road improvements more publicity, since if the public was made more aware of them, it might be a first step towards convincing the politicians of the absolute necessity of implementing them at the earliest opportunity.

There was certainly an antipathy to viaducts—Silvertown Way, which was largely

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[14] "Preliminary Studies for an Additional Bridge across San Francisco Bay." Department of Public Works, State of California.

on viaduct, could be taken as an example. Economic considerations had forced the engineers to build the viaduct and leave the surrounding development just as it was before. Owing to the need to save money, a major road proposal involving a number of viaducts might be shorn of its proper setting.

The Author made the point that the construction of the ring road would involve the rehousing of about 20,000 persons which "could probably be accepted without undue difficulty." It was well known, however, that the County of London had an overspill population problem involving about 300,000 people. The Council were trying hard, under the expanding towns provisions, to come to some arrangements to deal with that problem, but the negotiations were not going so rapidly as they would desire, and there was a fear that as they moved people out more came in.

Mr J. L. Paisley (Ministry of Transport and Civil Aviation) observed that the term ring road was perhaps unfortunate for whilst near the centre of any large city a complete ring might in fact be called for, that was not inevitably or invariably so. Justification was required for each of the individual links which might go to make up a ring. On that basis the conflict a number of speakers had mentioned between ring road and radial was not so real. The centre of a large city was not a Euclidean point.

Mr Paisley joined with Dr Glanville and other speakers in asking for more data in particular cases. In the analysis of that data a technique observed in America could produce a revealing picture. On the basis that traffic would, if it could, proceed in straight lines from origin to destination what were termed "desire lines" were plotted on the map. The examination of those lines could throw traffic flows into vivid relief and illustrated where the ideal solution lay.

The Author, in reply, observed that Dr Glanville knew America well, but his own knowledge was second hand, so that he might have been rash to suggest that the ring-road principle was inapplicable to grid-pattern cities. Although Dr Glanville did not agree with that, and might have better grounds for his view, the Author still felt that a ring road was of less value for, and to give useful results was less applicable to, a grid-pattern city rather than an irregular radial type such as those in the United Kingdom and in many other parts of Europe.

In regard to Dr Glanville's comment that proposals involving large financial outlay were inacceptable to the government, that was thought to have been true at the time that the London County Council formulated the proposals in its development plan. The government had not felt able to provide its share of the money for the "A" ring or, to put it in another way, did not feel that the necessary part of the national resources could properly be diverted to it at that time. It would have been unwise for the Council to have launched a series of similar proposals in the development plan individually of the same magnitude as the ring-road proposal. If it had not been realistic there would have been far greater government criticism of its proposals, greater reconsideration, and more confusion in the public mind. Whether the same project if raised again today might find a more favourable reception he did not know.

On the question of the speed expected on ring roads, he thought in writing such a Paper it was undesirable to err on the side of optimism, because when dealing with vehicles travelling at different speeds it was inadvisable to put too high a value on the potential average speed, bearing in mind that the new road, if constructed, would be running fairly full. On the other hand, the road would have been designed for a safe maximum speed of 70 m.p.h.

As to data, Dr Glanville's comments were applicable so far as information available at the start of the traffic investigation was concerned but sufficient further data had been obtained during the inquiry to enable reasonably firm estimates to be made. Finding the time and therefore the manpower required for analysis had hitherto been a much greater difficulty than finding them for collecting data; the advent of electronic machines should enable much more to be done in that important sphere. The Author, however,

was not inclined to ignore the "hunches" of persons of considerable experience in such matters because he thought that they were often of great importance.

He was pleased that Mr Andrew had pointed to what seemed the most cogent argument for the construction of such a project, namely, its comparative cost in relation to surface improvements giving the same relief. Mr Kerr-Nesbitt had also made a very useful contribution in presenting a point of view different from that of the engineer and giving some sound advice on how to get the projects carried out. It should be borne in mind, however, that politicians were responsible for spending the national revenue in a suitable manner and sharing out an inadequate sum of money for various needs. It might sometimes, for instance, be necessary to choose between expenditure on defence and expenditure on road improvements.

Mr Andrew might be on firm ground in suggesting more than one ring for an area with a population of 8,000,000 but the Paper attempted to assess the value of one only. That in itself was a difficult enough problem but its complexity would be greatly increased if other ring roads were to be considered in addition. He agreed that, if they were provided, their main function would probably be as Mr Andrew had described but, if so, there was no particular reason why such roads should be continuous rings. The North and South Circular Roads fulfilled such a function but were not continuous. The Author did not think that his third conclusion to Part I conflicted with Mr Andrew's views. With regard to the normal type ring roads referred to on p. 116 the County of London Plan included one round the central area, and the sole purpose of referring to one in the outer position was to make possible comparative estimates of both complete systems. The figure of £230,000,000 therefore included a "B" ring road of normal high-standard surface type, but his own estimates and those for property acquisition to which he had access showed that, contrary to Mr Andrew's suggestion, the motorway type of ring in that position would cost about 50% more.

Mr Clayton's comments were useful in amplifying and underlining various features of the Paper, presentation of which had been precluded by lack of space.

Mr Clayton stated that a ring road might possibly be the most economic if not the most effective system. That brought out a point which the Author wished to underline. He had not said that the ring-road conception was necessarily the answer to London's traffic problems, and he would not like it to be thought that he did not appreciate the very considerable amount of valuable work which Hounsfield had done.

Dr Charlesworth had raised several interesting points. The Author was rather surprised that there was evidence that radial traffic in America formed a fairly constant percentage of all entering traffic irrespective of the size of the town. It did not agree with the considerable and consistent evidence which he had studied. Moreover, there was no doubt what the percentage was in the case of London. He agreed that radial traffic was important but the proportions of radial to ring-road traffic shown in Fig. 9 were not typical. Four of the main access points would have heavier volumes on the radials, four on the ring road, and one would be about evenly balanced. An appreciable part of the radial volumes consisted of traffic which would prefer to use the ring rather than surface streets in the central area, but such traffic would tend to concentrate on the inner fringe of the radials in accordance with the principle illustrated in Fig. 8. He estimated that on 1954 volumes the net relief to the central area would be about 32,400 vehicle-miles/hour and he considered that that relief would reach its maximum throughout the core of the central area where it was most needed. Apart from the effect of the internal traffic using the ring he did not think that the distribution pattern would change in essentials. On intersection designs he wished to remind Dr Charlesworth and other speakers that Fig. 9 was a volumetric diagram applying the roundabout-cum-fly-under principle. It happened to be the second most heavily loaded intersection in the series so far as the roundabout portion was concerned and the most heavily loaded of any in total traffic. Although the Author thought that Dr Charlesworth tended to attach too much significance to the discomfort and accident potentials at any roundabout, he agreed that in the case of that intersection, and possibly of one or two others, an increase in weaving capacity of the roundabout section was desirable and that that aspect should be borne in mind if and when

detailed design was undertaken. It might well be that more elaborate treatment would be useful but it should be remembered that the extra cost would be relatively high and a balance might have to be struck. The loading of the radials consequent on entering and leaving the ring would demand similar consideration. It might well be that, for instance, instead of a three-lane road, as at present proposed for an improvement which was being carried out on a piecemeal and rather slow basis, a four-lane road might have to be envisaged for some distance from the access point. On total vehicle mileage he was aware of the Road Research Laboratory estimate and when allowances had been made for the slightly different bases on which it and his own estimate were formed, he thought that it confirmed that his own was of the right order. He agreed that, as a matter of prudence, he might have erred on the side of under- rather than overestimation. The ring-road speed used in calculating journey "areas of influence" (Fig. 8) was 30 m.p.h. He thought that Dr Charlesworth's feeling that the potential use of the ring road had been, if anything, underestimated, was based on a slight misunderstanding of the abbreviated argument in the Paper when he applied a figure of 41% to radial traffic. That percentage related to that proportion of eligible vehicles in *all* traffic in the fringe area; that traffic was composed of through (established by the survey as being 20% of the whole), radial, and internal/local (together totalling 21%). The survey, however, had not provided any data on the percentage contributed by the last named. A study of the effective fringe area built up by superimposing successive "areas of influence" on each other and related to the total estimated volumes and densities of that group showed that the proportion likely to use the ring road would be small, probably less than 1,500 vehicle-miles/hour. That uncertain and relatively small volume would, however, be offset by the proportion of eligible traffic which for various reasons might not use the ring and, in order not to overestimate the usefulness of the ring road, the Author had decided to ignore the net figure.

On the question of the values of vehicle-hours (a point raised also by Mr Austin) the figure of 21·5*s*/hour for a car was based on two main factors, namely, the value of driver's and passengers' time and standing charges. The first was the more important but unfortunately more difficult to estimate, but he had assumed 1·75 person per car and 10*s*/hour per person, giving 17·55. That was admittedly arbitrary but was based on the fact that $3\frac{1}{3}$% of the population (equivalent to 10% of the working population) had incomes exceeding £1,000 per annum, at which the value of an hour of working time was 10*s*. That seemed reasonable after balancing the lower value of the small proportion of professional drivers against the higher time value of persons in income groups exceeding £1,000. That figure might have to be revised in the light of data collected by the Road Research Laboratory, but he was inclined to think that no significant difference would emerge. Standing charges included depreciation (based on actual costs of twenty-six specific makes in five horse-power groups up to 2,250 c.c. and on an assumed average cost of £1,500 for cars of higher horse-power, weighted according to the number of licences issued in London), tax, insurance, garaging (average of £26 per annum), and running time (600 hours per annum). Those totalled 4*s*. Maintenance had been included under running costs.

The Author was tempted to agree with Mr Andren that traffic needs might not be considered in their correct perspective by planners, but it should be remembered that planning in that sense covered so many aspects, some of which might be of even greater importance in certain cases, that it would be unreasonable to expect specialist knowledge on all of them. Road-traffic engineering was extremely complex and he hoped that he had said nothing which would lead planners to think otherwise. The proposed ring road would not consist of a uniform number of traffic lanes but there was little scope for variation. He had been careful to say in his introductory remarks that he did not claim that the ring-road principle would have advantages over all other systems. He thought that Hounsfield's grid system had much to commend it but he had made no such comparative study of its traffic value as he had for a ring road. It would, however, probably be much more costly. The estimate of 18,000 vehicles entering the inner area hourly in 1937 was based partly on the same information as was available to Hounsfield, hence the similarity. He found some difficulty in following Mr Andren's comments about tangential traffic

but he agreed that the ring road was, in a sense, a sum of a large number of relief routes on a common alignment. He had, in reply to Dr Charlesworth, commented on the design aspect of the more heavily loaded access points, but the figure of 500 vehicles/hour/lane was much exceeded in various parts of London and he hoped that in most cases six lanes would suffice for the radials. He agreed that a desirable corollary to a motorway ring would be motorway radials beyond it. Inside the ring improvements should, ideally, take place in the following order:—

(1) Intersections on the main radials.
(2) Main radials.
(3) Secondary radials.

He was pleased that Mr Austin agreed with the usefulness of trying to assess the economic value of traffic improvements. It was true that other projects—not necessarily roads—might show a better return, but the basic fact was that the United Kingdom could not exist without roads. It was also true that the present road system was obsolete but the extent to which it could be modernized was governed by the proportion of the national resources which could be devoted to it.

It was difficult to put a realistic value on the cost of traffic delays and in trying to assess that value one had to compare present conditions with some arbitrary standard. If present average speeds could be increased to, say, 20 m.p.h., reduction in delays in the central area might save the community about £10,000,000 a year but he would not like to hazard an opinion of the cost of delays in London as a whole, except that it was clearly considerably more than in the central area alone and Mr Austin's figure might well be on the low side. Mr Austin had asked whether caisson construction had been considered for the under-river section. It had been, but not in detail because the use of caissons would produce considerable maintenance difficulties and in any case would have had no appreciable effect on the total cost. He asked to be excused from further comment on that feature because if and when a project such as the ring road was carried out a Paper on it would be of extraordinary value and interest and it would be premature to anticipate it. He noted Mr Austin's reservations on lane capacity but, whilst agreeing that a broken-down vehicle, weaving, and the mixture of traffic were adverse factors to maximum traffic flow, he would point out that some much higher rates of flow than those assumed for the purpose of the Paper had been observed on many occasions and he held to his estimates as being reasonable but perhaps on the conservative side.

He welcomed the contribution from his valuation colleague, Mr Kerr-Nesbitt, partly because the cost of property acquisition for street improvements in the inner area of London was frequently three times that of the works and partly because his views on publicity, the antipathy to viaducts, and the re-housing problem helped to paint a more complete picture of what the project involved than if the traffic engineering aspect alone were considered.

Mr Paisley's reference to one American device for analysing traffic movement was of interest and could be applied, for instance, to diagrams such as Fig. 6. He thought, however, that the "area of influence" method as shown in Fig. 8 achieved the same object in that particular case with less labour.

The closing date for Correspondence on the foregoing Paper was 15 March, 1956. No contributions received later than that date will be published in the Proceedings.—Sec.

MARITIME AND WATERWAYS ENGINEERING DIVISION MEETING

3 January, 1956

Sir Arthur Whitaker, Vice-President, Chairman of the Division, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Author.

Maritime Paper No. 31

RECONSTRUCTION OF THE GALLIONS LOWER ENTRANCE LOCK AT THE ROYAL DOCKS OF THE PORT OF LONDON AUTHORITY

by

* John Alfred Fisher, M.I.C.E.

SYNOPSIS

The Gallions Lower Entrance was built by the Engineering Department of the London & St Katharine Dock Company with direct labour and based on the design of the Upper Entrance Lock by Sir Alexander Rendel and opened on the 20th July, 1886.

The barrel of the lock has a curved invert to a radius of 110 ft. The principle work of the reconstruction consisted in the removal of the haunches of this invert and the substitution of reinforced knees. The opportunity was taken to recondition the walls and granite work and to provide new gates and machinery.

To enable the base of the lock to be excavated and new reinforced sections inserted, a ground-water-lowering system was provided, and the invert of the lock was loaded with concrete kentledge. The inner dam was of blockwork and built underwater by divers. The outer dam was of steel sheet-piling and filled with rubble.

All the granite work has been replaced, much of the original material being re-used. The brick facing of the walls was removed and, apart from the gate recesses, the lock was refaced with concrete throughout. Emergency sills were provided at the inner and outer gates. The three original culverts in the floor of the lock were removed and reconstructed to larger dimensions. This entailed the diversion of high-pressure sewerage pipes, together with gas, water, and other services.

The original gates have been replaced with buoyant gates and the gate-operating machinery is of a double-geared balanced type driven through fluid couplings from an electric motor. The sluice machinery has been rebuilt with the original rams reconditioned for use with oil. The outer sluiceways were found to have been the cause of extensive scour on the outer apron. By means of a tidal model this scour was investigated and the sluiceways have been modified.

The jetties have been reconstructed and the opportunity has been taken to build one section of a flexible steel type. This work is not yet completed.

It is expected that the whole of the reconstruction work when completed will have taken approximately 5 years, and the total cost will exceed £1 million.

Introduction

The Gallions Basin at the Royal Docks of the Port of London Authority has two entrance locks known as the Upper and Lower Entrances. The works described in the Paper concern the Lower Entrance. (Fig. 1, Plate 1.)

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This lock is one of four lock entrances serving the Royal Group of Docks. Two of these locks are of shallow depth and are limited to barge traffic. Of the two remaining locks, the condition and limitations of the Gallions Lower Entrance has necessitated all vessels using this group of docks being handled for some time past by the King George V Entrance Lock. Investigation into the length, beam, and draught of these vessels revealed that, with minor improvements and reconditioning, a large number of these vessels could be satisfactorily dealt with at the Gallions Lower Entrance, and would relieve the heavy duty being undertaken by the King George V Entrance.

The Royal Group of Docks consists of the Royal Victoria, Royal Albert, and King George V Docks, having a total water area of 235½ acres and containing berths for fifty-two large vessels.

This Lower Entrance Lock was built by the Engineering Department of the London & St Katharine Dock Company, employing direct labour under their engineer, Mr Carr, and was based on the designs of the Upper Entrance made by Sir Alexander Rendel.

According to the Minutes of the London and St Katharine Dock Company, 1884 (now in the possession of the Port of London Authority), the construction was authorized in April 1884, completed at a cost of £328,830 12*s* 8*d*, and opened to shipping on the 20th July, 1886. The original works are, therefore, about 70 years old.

Longitudinal sections and cross-sections of the lock before and after reconstruction are given in Fig. 2, Plate 1, from which it will be seen that throughout the barrel of the lock the invert is at a radius of 110 ft. This cross-section was no doubt suitable and convenient for the vessels of those days, but modern vessels with a very small rise of floor can be considered as practically flat-bottomed. The length and beam of the lock, 550 ft × 80 ft, are in reasonable proportion and adequate for a large number of modern vessels, but the limitations imposed by chain-operated gates combined with a curved invert were appreciable. If the depth at the gate sills could be maintained throughout the lock those limitations would be largely removed. It was, therefore, decided to remove the haunches of the invert by the substitution of reinforced knees, and this forms the principal reason for the reconstruction of the entrance. The opportunity was taken also to re-condition the walls of the granite work and to provide new gates and machinery.

Design

Plans and estimates were prepared for the work in 1951 by the Chief Engineer of the Port of London Authority. Owing to the nature of the work proposed and the fact that the lock was at that time in use, the extent of the repairs and the condition as to the original work could not be accurately ascertained. The information available of the existing construction was very limited, and no major repair works had been carried out which would have assisted in checking the accuracy and condition of the original construction. The brick facing of the walls was known to be loose and instances had occurred of the brickwork falling away. Where it could be seen, the face of the brickwork was considerably worn and eroded except in the gate recesses. This wear and tear was undoubtedly attributable to the extent of barge traffic.

In consequence of the loose nature of the brick facing, its thickness was ignored for the purpose of stability calculations during that period of construction when the work would be in the dry. Borings were taken through the bottom of the lock, which proved that the original drawings were approximately correct, but the thickness of brickwork and concrete appeared to be rather less than expected. This

information, combined with the reduced value of the wall thickness, gave rise to some anxiety in the design of the concrete haunches or knees necessary to obtain the increase in depth.

The original Cornish granite was worn at the square and hollow quoins and on the sills, but otherwise thought to be sound. The granite sills were at a radius of 105 ft. The new sill was to be straight with the apex set back 6 in. and in line with the redressed hollow quoin. As much as possible of the granite sill was to be left undisturbed, renewing the apex stone, adjacent stones, and wherever damaged. On the hollow quoins and square quoins approximately 20% of new material was provided for the renewal of worn or damaged stones, the remainder to be left undisturbed and dressed to the new face or curvature. The hollow quoins, originally of 13-in. radius, were to be redressed to 14-in. radius throughout and to the outline as shown in Fig. 3, Plate 1. Where granite stones were removed or renewed, anchorage bolts were to be provided as shown in Fig. 4, Plate 1. These 1-in.-dia. anchor bolts were of stainless steel and provided with a stainless-steel plate washer and nut and grouted in after the stone was fixed.

The brickwork facing of the walls was to be removed throughout and replaced with a granite-concrete facing keyed into the original concrete, with anchor bolts and with B.R.C. mesh fabric, and to be continuous from the new concrete haunch to the coping. Steel rails were to be set into the concrete at coping level to withstand wear and tear and to prevent spalling.

The original gates and hydraulic machinery were 70 years old and were to be replaced with modern semi-buoyant and welded steel gates and electrically operated machinery. Provision was made for new gate pintles and new anchorages. The original roller paths were to be removed. At the inner and outer gates new sills and hollow quoins were to be provided for emergency purposes, into which the existing gates could be placed and which would enable the lock to be temporarily pumped down for an urgent repair should occasion arise.

The existing sluices were to be re-conditioned and sluice machinery overhauled.

A new a.c. 6,000-volt, 3-phase supply was to be provided from which the 415-volt and 240-volt services are taken.

The entrance jetties were to be repaired as required, and new administration and operating buildings included.

Contract

Competitive tenders were obtained and the contract was awarded to Messrs John Mowlem & Company Ltd, on the 25th September, 1951. Certain features of the contract should be mentioned.

The contractor was called upon to indemnify the Port of London Authority against damage caused by a failure of the dams, which would jeopardize the whole of this group of docks, and would cause very serious damage to the vast quantity of shipping likely to be present. There was direct water access to the dams from both dock and river, so there was a "third party" risk involved; failure could arise from a large vessel striking a dam. Both the inner and outer dam were therefore constructed with the maximum possible safeguard against accidents of this nature. The cost of these dams forms an unusually high proportion of the contract sum.

To facilitate the removal of the toe of the walls and the construction of the reinforced concrete haunches or knees, the contractor was called upon to provide a system of ground-water lowering and to provide emergency stand-by electrical supply to avoid the danger of any failures due to a breakdown in the public electricity

supply. As an additional safeguard against upward pressure from below the floor of the lock should the ground-water lowering fail to be fully effective, the contractor was called upon to install kentledge to load the base of the lock to 10 tons/ft run prior to dewatering the lock.

Site investigation

Thirty-nine boreholes were made over the site. These include bores taken by the contractor for the site of the dams and twenty of them were holes to take deep-well pumps for the ground-water lowering. Five bores were taken through the sill of the lock, including two at the gate platforms.

The chalk level was found to be variable and lay within 10 ft of the invert. Overlying the chalk is a bed of sandy gravel which varies in thickness from 13 ft up to 36 ft. Above the gravel the material is generally of poor quality backfilling with soft clay and peat. Tests on samples of chalk were carried out and also tests to measure the angle of friction between the concrete and gravel. These tests show that with the blockwork type of dam the resistance to sliding friction would be insufficient for safety, and in consequence the toe of the dam was extended under water by a concrete apron abutting on to the existing brickwork of the original construction. Deep sounding tests were made of the gravel below water level. Some difficulty was experienced in penetrating this gravel from a floating stage. From the result obtained it was estimated that the penetration resistances of the $1\frac{1}{2}$-in.-dia. cone at depths of 7–9 ft in the gravel were between 2,500 lb/sq. in. and 4,000 lb/sq. in. The weight of saturated mud was found to be 90 lb/cu. ft and of saturated gravel 135 lb/cu. ft.

From the result of the deep bore tests the angle of friction was taken as 40°. To determine the resistance of the blockwork dam to sliding, tests were made under submerged conditions. Loose gravel was placed on to a concrete slab within a square shear box and subjected to vertical pressures, and the results were plotted. The angle of friction was found to be 36°. With a smooth mortar block the angle of friction was 33° and on smooth precast concrete under 3 ton/sq. ft vertical pressure the angle was 32°.

Tests were made upon the chalk. Preliminary samples showed the material to be fissured and the chalk surfaces in the fissures were soft. Triaxial compression tests were made on samples 4 in. in diameter by 8 in. long under applied pressures. The average cohesion or shear strength was found to be low and between 40 and 50 lb/sq. in. on undrained samples. Deep sounding tests were made and achieved cone pressures ranging from 600 to 900 lb/sq. in., corresponding to shear strengths of 60–90 lb/sq. in., indicating that the undisturbed shear strength was considerably greater than that indicated from the original samples.

Work commenced at the end of 1951 in the dredging of the entrance, which was attended to by the Port of London Authority without cost to the contractor. On 20th January, 1952, the site was handed over to the contractor, who immediately erected a Bailey bridge of 90-ft span to give access to the island site. This island site, the main working area for the contractor, was first used as a blockyard for the manufacture of the blocks for the dam and for kentledge (Fig. 13, facing p. 154).

It had been the contractor's intention to use a concrete-block gravity dam for both the inner and outer positions. During the investigation which took place simultaneously with the construction of the blockyard, the contractor decided against the use of a blockwork dam for the river or outer position owing to the presence of a

large scourhole immediately beyond the apron, and received approval to substitute a steel sheet-piled dam with hardcore filling and berms.

Ground-water lowering

It had been decided in the contract that the contractor was to provide a system of ground-water lowering by means of deep-well pumps to reduce the hydrostatic forces on the walls and floor of the lock. To prevent any uplift in the invert when the lock was emptied, it was necessary to reduce the hydrostatic uplift to less than the deadweight of the floor. This required the head of water to be reduced to a level of *+ 82·0 W.D., but to provide a margin of safety it was aimed to reduce the head to a level of + 72·0 W.D. Such a reduction would also result in a reduction of pressure on the walls. The contractor also took the opportunity to provide deep wells in the dams to reduce uplift and to permit the development of the full sliding resistance at the base.

The natural conditions of the site were suitable for deep well installations and the pumping took place in the layer of gravel overlying the chalk. The original standing water level was at + 102 W.D. so that the water-lowering problem was to provide sufficient wells to reduce the head in the gravel to + 72·0 W.D., and to place them so as to maintain a satisfactory level under the whole of the lock floor.

The arrangement adopted was to provide ten wells at approximately 80-ft centres each side of the lock and approximately 50 ft from the lock coping, clear of permanent works. The positions of these wells, together with the wells for the dams and for observation purposes, are shown in Fig. 6, Plate 2. Altogether thirty-four wells were sunk.

During the boring of these wells further information on levels and permeability was obtained. This information confirmed the original assumption and the installation was continued as planned, except that well No. 7 did not yield any appreciable quantity of water and was not provided with a pump. Well No. 19A was added at the river end of the south side to provide additional pumping capacity at this critical point when pumps had to be removed for maintenance. (It is believed that well No. 7 is on the site of an old pumping sump used during the original construction of the lock. Some wood was found during the boring of this well and of well No. 6.) During test pumping the level of water in No. 7 well was not reduced, as in the other wells, until after the lock had been drained.

The stability of the dams constructed at the ends of the lock required that the maximum resistance to shear should be developed in the gravel, and for this purpose a row of four wells, numbered 31 to 34, was provided in the outer hardcore-filled dam to reduce the water level within it, and a similar row of five wells numbered 25 to 29 was bored between the cut-off piles and the toe of the inner block dam.

The borings for the wells were 24 in. dia. reduced to 18 in. dia. in a few cases in the chalk. The permanent well linings were of 12-in. steel tube perforated and wrapped with $\frac{1}{16}$-in. steel mesh in the region of the gravel, and with a 7-ft plain sump beneath this in the chalk (Fig. 7). The space between the boring tubes and well lining was filled with graded filter gravel up to the level of the top of the gravel zone.

Submersible electric pumps of approximately 12 h.p. were placed in the wells on 4-in.-dia. riser pipes. These were connected to a common 8-in.-dia. pumping main on each side of the lock and could discharge either into the River Thames or into the dock.

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* W.D. Works Datum=100 ft below Newlyn Datum.

The work of boring the twenty wells along the sides of the lock was started in December 1951 and completed in about 3 months. Generally the boring was continued about 7 ft into the chalk, which gave depths between 63 and 87 ft, with an average of 71 ft. In six of these wells 18-in. tubes had to be used because the

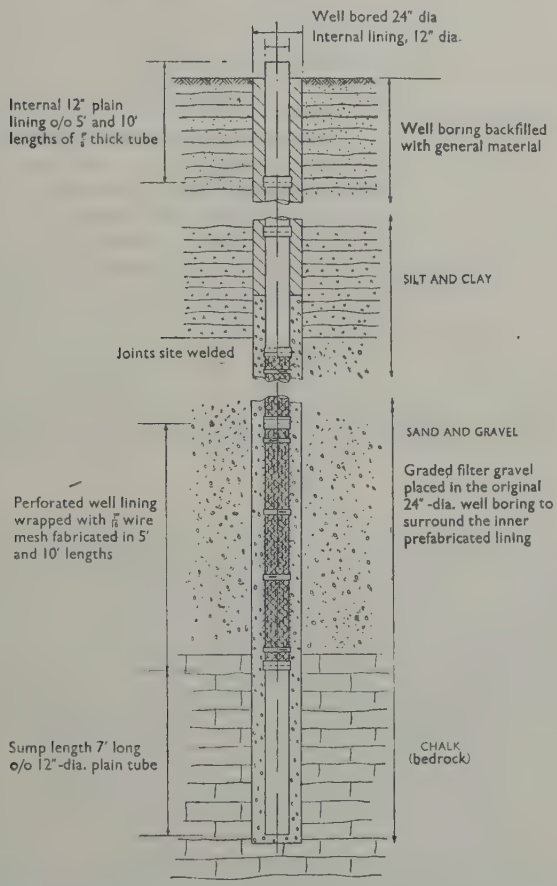

Well bored 24" dia
Internal lining, 12" dia.
Internal 12" plain lining o/o 5' and 10' lengths of ⅜" thick tube
Well boring backfilled with general material
SILT AND CLAY
Joints site welded
SAND AND GRAVEL
Graded filter gravel placed in the original 24" -dia. well boring to surround the inner prefabricated lining
Perforated well lining wrapped with ⅛" wire mesh fabricated in 5' and 10' lengths
Sump length 7' long o/o 12"-dia. plain tube
CHALK (bedrock)

FIG. 7.—TYPICAL 24-IN.-DIA. WELL. CONSTRUCTIONAL DETAILS RELATIVE TO TYPICAL SITE DATA

friction against the 24-in. tubes was found to be too great for the equipment available.

The 8-in. pumping main was of steel pipe with vitaulic joints and could not be completed until July 1953 owing to the delayed progress of the dams.

Five observation wells were bored into the gravel. The position of these was governed by the existing buildings and future construction. One hole was bored through the bottom of the lock, using a diamond drill and working from a staging off the temporary bridge.

The electric cables from each pump were run alongside the pumping main for

protection and those from the south side passed over the temporary bridge. Elsewhere they were protected by steel tubes and laid in the ground. Great care was taken of these cables to prevent them being damaged during the progress of the contract. The controllers and indication for the pumps were housed in a building which also contained the stand-by electrical supply.

The main electricity supply was fed through a protective circuit-breaker and three changeover switches to three distribution boards, one for the even-numbered pumps and one for the odd-numbered pumps, the third being available for the pumps in the dams, etc. To ensure consecutive starting of the pumps, the starters were connected to a process timer, which continually operated all starter buttons in a 90-sec cycle. If a starter tripped out because of a current overload, the circuit to the process timer was broken, leaving the starter to be inspected and operated by hand; but if the cause was a current failure the pump would be automatically restarted.

The public electricity supply was used for the normal running of the pumps, but to safeguard against failure duplicate diesel generator sets were installed, each of 375 kVA capacity. Each set was capable of running the full equipment. Manual operation was required to start these generators and to operate the main changeover switch gear, and an electrician and mechanic were in constant attendance. These sets were subjected to regular test and trial runs were made at peak-demand periods.

It was very important to reduce any period of stoppage to the minimum, since the water lowering was largely the reduction of artesian head in a fairly permeable gravel, and would build up rapidly once the flow of water from the wells had ceased.

Inner dam

Before the inner dam could be constructed it was necessary to provide protection against shipping and craft within the dock. For this purpose the contractor drove a steel protection formed with 12-in. × 12-in. broad-flanged beams driven in 8-ft trestles, 20 ft apart, and framed in both directions above dock water level. These broad-flanged beams were pitched from a floating pile-frame and driven into the chalk. Half timber fenders were secured to these trestles and walings. Approximately 120 tons of steelwork was used for this protection. Advantage was taken of this structure to carry equipment for the construction of the dam foundations and to house the divers working on the block setting.

No. 3 Larssen steel piling was next driven from two electric derricks mounted on the inner ends of the walls of the lock. This steel sheet-piling was 5 ft clear of the vertical face of the blockwork dam; it was required to retain the mud at the dock bottom and to enable the excavation of the dam foundation to be carried out. The excavation of mud overlying the gravel was by grabs to approximately 10 ft below the dock bottom, after which the concrete forming the bed for the first course of blocks was placed by skip in bays 16 ft × 44 ft. This concrete bed was between 12 and 18 in. in thickness. The excavation, levelling-off, and placing of concrete took, on an average, 7 days per bay for the thirteen bays covering the base of the dam.

At each end of the dam, and adjoining the lock wall, the foundations were stepped up one course. The first course of concrete blocks was carefully laid by divers on this prepared foundation, and clay was laid between the face of the dam and the steel sheet-piling. The first layer of blocks having been completed the progress of blocklaying speeded up considerably. The average progress of blocklaying in the bottom course was two per day. The average over the 1,350 blocks in the dam was fourteen per day, with a maximum of twenty-five. Six divers were

continuously employed on this work, and the accuracy in block setting achieved by these men was remarkably good.[1]

The height of the inner dam is 48 ft and the depth of water 42 ft. The overall length is 207 ft and the total weight of the dam is 15,000 tons, with an average of 72 tons/ft run. Each standard block weighs 11 tons and contains $6\frac{1}{2}$ cu. yd of concrete. Special blocks were cast for the base course and at the corners. The lower four courses, including the base course, have a width of $3\frac{1}{2}$ blocks, making 31 ft 6 in. Course No. 5 is 3 blocks wide (27 ft); course No. 6 is $2\frac{1}{2}$ blocks wide (22 ft 6 in.); course No. 7 is 2 blocks wide (18 ft); courses Nos 8, 9, and 10 are $1\frac{1}{2}$ blocks wide (13 ft 6 in.); and courses Nos 11 and 12 are one block wide (9 ft).

All blocks are interlocked on the horizontal bed, and a grouted seal is provided on all face joints on the pressure side. The blocks for this dam and for the invert kentledge were precast in the blockyard and stacked for curing. The concrete was of ballast with an 8 : 1 mix, and the average strength was 2,500 lb/sq. in. at 28 days.

All the concrete for this work was mixed in one $\frac{1}{3}$-cu.-yd "Cumflow" mixer which achieved an average output of 12·26 cu. yd/hour, or nearly 100 cu. yd/day of 8 hours.

The blockyard was completely traversed by a travelling electric tower derrick of 15 tons capacity with a 120-ft lattice jib.

Despite the weight of the dam the frictional resistance between the concrete and the gravel did not provide a sufficient margin of safety against the total thrust, and accordingly it was decided to place underwater concrete between the inner toe of the dam and the brickwork of the original lock apron. This concrete was eventually to form part of the emergency sill and permanent work of the contract. Particular care was taken with this section of the work and each section of excavation was inspected underwater by the Resident Assistant Engineer.

The concrete is 10 ft thick and was placed in 15-ft bays, for which special shuttering was prepared. A 3 : 1 mix was used with $\frac{3}{4}$-in. all-in ballast. It was placed with a 6-in. tremie pipe, the lower portion of the pipe containing a plug. This plug was forced out by the weight of concrete and prevented any inflow of water into the tremie. Once started the concrete was poured continuously until the whole of the shuttering was filled. After the lower concrete was placed the tremie was maintained 3 ft below the concrete level until the finish. This concrete was finished higher than the required level for the contract, and subsequently the top 12 in. was removed by compressed-air tools, in the dry.

Core borings later taken into this tremie concrete proved the material to be of excellent quality and satisfactory for retention.

Outer dam

The outer dam was constructed of two skins of steel sheet-piling approximately 40 ft apart. The outer skin was of No. 3 Appleby Frodingham piling in 70-ft lengths. The top of this sheeting was 18 in. above the highest known tide, and the sheeting was driven approximately 15 ft into the chalk. At the centre of the lock the chalk level was lower because of the scour which had taken place. At this position the minimum penetration was 10 ft. The sheet-piling was returned at each end to meet an original timber cofferdam. This timber cofferdam had been used for the original construction of the lock, and the portions of it which remained were in remarkably good condition.

[1] J. R. Malcolm and J. A. Lewis, "Civil Engineering Construction Under Water." Proc. Instn Civ. Engrs, Pt III, vol. 3, p. 738 (Dec. 1954). See p. 758 and Fig. 8, Plate 1, and Fig. 17, facing p. 753.

The inner skin of the dam was formed of No. 3 Larssen steel sheet-piles in 65-ft lengths. The top level of this sheeting was 15 ft lower than the outer skin.

The two skins of the dam were connected by sets of walings and tie-rods. The bottom walings, which were situated just above low water level, were of 12-in. × 12-in. broad-flanged beams, with twin 2½-in. M.S. tie-rods at 10-ft-6-in. centres. The middle walings were of double 12-in. × 3½-in. channels with single 2½-in.-dia. tie-rods at 10-ft-6-in. centres. The top walings were of double 9-in. × 3-in. channels and connected with 1¼-in.-dia. tie-rods at 10-ft-6-in. centres.

Four 21-in.-dia. pipe sluices were constructed through the dam at low water level.

The sheet-piling was driven from two electric derricks situated at the extreme end of the lock walls. These derricks also placed the walings and tie-rods, and were used with grabs to excavate the mud from between the sheet-piling preparatory to the placing of hardcore. The filling between the sheet-piling and on the outer berm was of brick rubble and concrete hardcore, the majority of which was obtained from works at the London & St Katharine Docks.

The inner berm was composed of granite sets, a large quantity of which was also readily available. The total volume of hardcore filling employed was 30,000 cu. yd.

Immediately inside the dam the contractor installed one 10-in. electrically driven centrifugal pump, and two 6-in. vertical-spindle self-priming centrifugal pumps.

The construction of the dam took appreciably longer than anticipated being completed in July 1953. Pumping started at the end of July 1953, the deep wells having been completed about 8 months earlier.

The first tests were not successful owing to leakages at the shoreward ends of the outer dam. Blisters were driven and further tests proved satisfactory. With the inner blockwork dam some movement took place in the horizontal joints when the water pressure from the dock became effective, and a number of the face seals had to be remade. The tests were finally effective by the end of October, and from that time onwards the dams have been remarkably tight.

The deep-well pumping was given a full-scale test with twenty wells in October 1952 and proved equal to expectations.

The main work of dismantling and reconstruction could not commence until the satisfactory completion of the dams and deep-well pumping and the Contractor's programme was recast accordingly (see Fig. 14, facing p. 154).

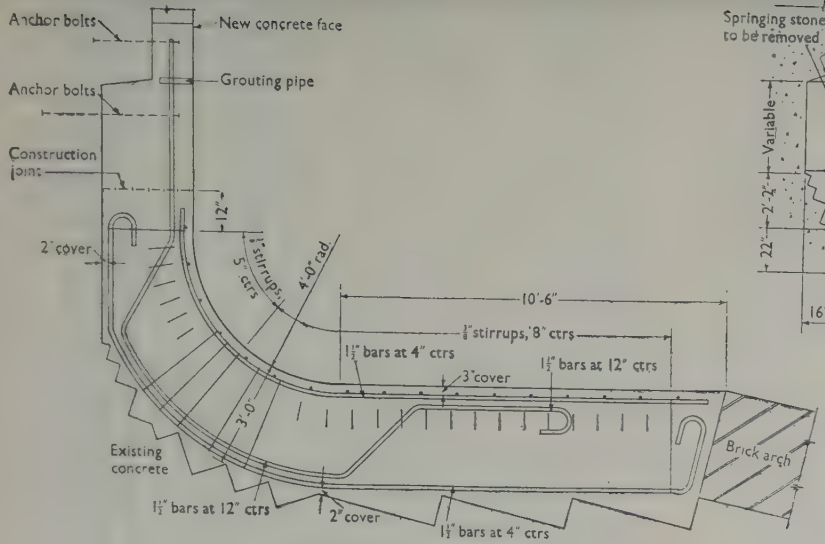
Knees

The most important feature of the new work was the removal of the brick haunches of the arched invert and their substitution by reinforced concrete angles or knees. This work involved a careful study of the forces in the structures, particularly during the conditions when the lock would be dewatered and the ground-water lowering was effective. Investigations were made to prove the thickness of brickwork and concrete. The removal of the invert haunch had to be undertaken with care, for in this position the stresses from the walls against overturning are at the maximum.

The work was specified as for underpinning in that the contractor was instructed to limit his excavations to lengths of 11 ft and to maintain a distance of 55 ft between excavations. The procedure agreed and adopted is shown in Fig. 8.

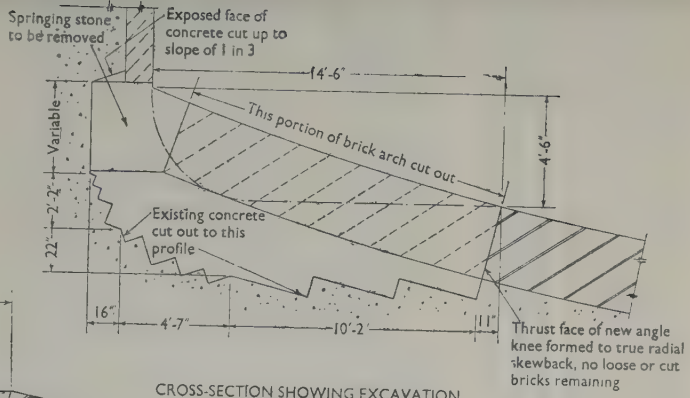
The condition of the material excavated was generally very good and the programme was maintained. After a few experiments had been carried out, a standard procedure was adopted as follows.

The area to be excavated was circumscribed with a Siskol 105 mining cutter



CROSS-SECTION OF NEW CONCRETE KNEE

Scale: ¼ in. = 1 foot


Springing stone to be removed
Exposed face of concrete cut up to slope of 1 in 3
14'-6"
Variable
This portion of brick arch cut out
4'-6"
2'-2"
22"
Existing concrete cut out to this profile
16"
4'-7"
10'-2"
11"
Thrust face of new angle knee formed to true radial skewback, no loose or cut bricks remaining

CROSS-SECTION SHOWING EXCAVATION FOR NEW CONCRETE KNEES

Scale: ⅛ in. = 1 foot

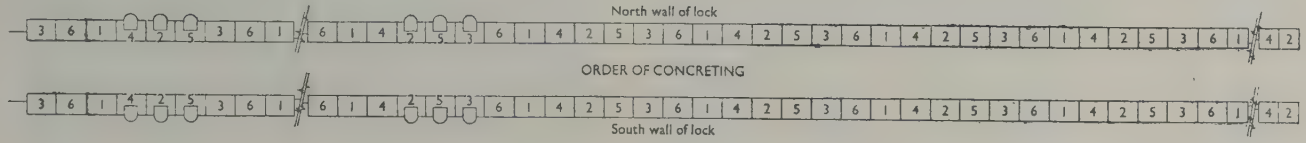


FIG. 8.—NEW CONCRETE KNEES

mounted on a 10-ton concrete block, which produced a gap approximately 4 in. wide for the full depth of excavation (i.e., up to 7 ft deep). Holes were drilled at 18-in. centres to a depth of 18-in. and ½-oz gelignite charges were inserted in each hole and fired in sequence. By this means the top layer was broken into pieces large enough to handle and the shock to the main concrete in the vicinity was very slight. After the removal of the upper layer, further charges were placed to within 12 in. of the final depth, and by this means the greater portion of the excavation was expeditiously removed. The final trimming of the excavation was carried out by pneumatic drills and the profile was carefully maintained.

The steel reinforcement (Fig. 15) was prepared in lengths of 11 ft and was placed by means of a derrick and carefully positioned. Each assembly of reinforcement weighed 5½ tons. Timber shuttering was adopted for this work and apertures were left in it for vibration. Four internal vibrators were used in each 11-ft length, and external vibrators were applied at the haunch. Quality "Eg" concrete (see section on "Concrete") was placed in one operation involving 25 cu. yd.

One of the principal features of the work was to ensure a tight fit and to ensure that the stress from the original wall could be adequately transferred to the new concrete without movement of the original structure. For this purpose special care was taken to avoid all possibilities of excess shrinkage. Various methods were considered, e.g., the use of expanding cement, or the insertion of hydraulic wedges. It was finally decided to rely upon the careful placing of the concrete, so that the first lift of concrete ended immediately below the shoulder of the excavation. After a period of 7 days this shoulder section was concreted, maintaining a 3-ft head, and well vibrated. Grouting pipes were set in each shoulder. After 3 months the grouting holes were drilled clear and pressure grouting was applied, using a 2 : 1 sand-cement mix with "Cebex 113" to which "Lissapol" was added.

Granite work

New granite was provided for the upper six courses at each hollow quoin and square quoin, and also for the apex stone and four other stones at each of the gate sills (see Fig. 16, between pp. 154 and 155). This material was supplied from Cornwall, and came principally from the Trevone, Trolvis, and Pelastine quarries of the Penryn Boss. A small amount was also supplied from the De Lank quarry of the Bodmin Boss. This material had been dressed at the quarries to shape and size.

Upon the removal of the granite stones of the hollow quoins it was found that every stone was loose and a layer of mud existed in the joints. This condition also existed with the square quoins. It was decided, therefore, to remove all the granite stones of the quoins except the base stone containing the gate pintle. The lower portion of the square quoin was found to be of sandstone and not granite. This was removed and replaced with granite concrete.

The material removed from the hollow quoins was generally sound and was redressed to the new radius and re-set. "Philbolt" anchors were used to key the backing of the granite stone to the original wall. The time taken to redress one hollow quoin stone was approximately 3 weeks, for this material was found to be harder than new stone of a similar nature.

It was not found practicable to redress the original stones of the sill in position, because the new sill shape would have involved so much cutting-away, so a trench was excavated on each side of the sill and all the original sill stones were removed up to the base of the hollow quoin. Apart from the renewal of the apex and the new stones provided, all the original sill stones were re-used and redressed. Stainless-

steel anchor bolts (Fig. 4, Plate 1) were used between each pair of granite sill stones. The whole of the granite work in quoins and sills was re-layed with $\frac{1}{4}$-in. joints, bedded in with 2 : 1 cement and sand, and grouted with neat cement. The hollow quoin stones were dressed in the stone-yard to within $\frac{1}{8}$ in. of their finished dimensions. This final $\frac{1}{8}$ in. was removed when all the stones were in position and the final measurements were taken.

All the work on the granite was carried out with pneumatic tools by eleven granite-masons with attendant labour.

Brickwork facing

The original lock walls were faced throughout with 13$\frac{1}{2}$-in. gault brickwork with an additional 13$\frac{1}{2}$-in. thickness at regular 5-ft intervals. This brickwork generally was known to be loose and had broken away from the concrete backing. Repairs have been carried out in the past by means of grouting in bolts with plate washers set flush with the face of the wall. The cavity between the brickwork and the concrete had entrapped water, creating a pressure head during locking with a tendency to loosen the brickwork still more. This was indicated by the presence of a layer of mud when the brickwork was removed.

It was decided to remove all the brickwork facing except that within the gate recesses (which was in blue brick), where vessels did not have contact with the wall, and where the appearance of the brickwork indicated it to be reasonably sound. The brickwork was removed by compressed-air tools and a heavy steel wedge or spade. It was found that where the additional thickness occurred the key into the concrete was good and the brickwork had failed in shear. Accordingly, certain of this work was left undisturbed and pockets, 4 ft × 1 ft 6 in., were cut to give access and key to the concrete of the main walls. The whole of the face was cleaned off and hacked to provide a good surface for the new concrete. Weak places in the original concrete backing were cut out and made good.

Holes were drilled at intervals of approximately 3 ft vertically and 4 ft 6 in. to 5 ft horizontally for the insertion of Philbolts. Altogether about 7,000 holes were drilled and Philbolts inserted and grouted-in prior to the concrete face being placed. These Philbolts are 1 in. dia. × 3 ft long, with 8$\frac{1}{2}$-in. × 8$\frac{1}{2}$-in. × $\frac{3}{4}$-in. plate washers and a tapered ferrule. The hole is drilled either 26 in. into the walls at an angle of 82° to the face, or 23$\frac{1}{2}$ in. into the wall at an angle of 60° to the face. The angle of 82° was chosen in preference to the horizontal to facilitate grouting.

Compressed-air drills with Tungsten Carbide steel bits giving a 2-in. clearance hole was found to be adequate for the maximum dimensions of the bolt insert. Each bit was used for about eighty holes (170 ft of drilling).

After placing the Philbolts the wedge was driven tight within the hole and the grout then poured. B.R.C. mesh reinforcement was fixed to the Philbolts at the correct distance before the shuttering was placed. The concrete facing (quality Eg with granite aggregate) was placed in 3-ft lifts and in lengths of 135 ft to 140 ft. Internal vibrators were used.

The test cubes achieved a strength of 2,500 to 3,000 lb/sq. in. at 7 days. Timber shuttering in sections 3 ft 6 in. × 5 ft 6 in. was used throughout for the walls.

Emergency sills and hollow quoins

As an alternative to the frequent practice of providing square recesses in the wall and sill, with the necessity for maintaining a floating cofferdam, it was decided to

provide emergency sills and quoins at each end of the lock, into which the gates could be fitted in the event of any future necessity to dewater the lock.

The outer emergency sill was on the site of the original apron. This apron was only about 5 ft thick and insufficient in depth. It was now necessary to excavate to the level of the original formation of the lock, and this entailed exposing the foundation gravel and chalk. It was undesirable to release any great quantity of water, and the level of excavation was 9 ft 6 in. below the water plane achieved by the ground-water-lowering system. The contractor accordingly decided to enclose the area of the excavation with chemical consolidation. A total of 347 boreholes was drilled around the area required in two lines at approximately 2-ft centres, to a depth of 2 ft below the base of the excavation. At the base of the walls where undercutting was necessary the holes were drilled at an angle and penetrated under the base of the original main wall. Chemicals were pumped into these holes in a progressive sequence, after trial had been made at a small sump at the head of the excavation. The chemical consolidation was by the Guttman process. It proved remarkably successful, and very little water was encountered.

The main excavation for the sill was then taken out in five sections, beginning at the centre. Each of these sections contained between 100 and 110 cu. yd. Drainage pipes were laid at the base of the excavation into the sump, and a six-way duct was inserted to communicate between each of the lock walls for future electrical supplies.

The whole of the concrete for each section was placed in one operation and occupied about 10 hours. The strength of the concrete test cubes at 28 days ranged from 3,655 to 4,475 lb/sq. in.

The concrete for the inner emergency sill had been placed during the construction of the inner dam and before the lock was dried out. This material was tested by core borings and the compression strength, 11 months after placing, reached 6,000 lb/sq. in.

When this work was undertaken it was not possible to complete the excavation and concrete adjacent to the lock walls. To prevent the undermining of the wall and the loss of gravel whilst this main excavation was in hand the contractor had driven sheet-piling 5 ft away from the toe of the wall. To complete the work for the inner emergency sill this sheet-piling had to be removed, and the excavation under the wall necessitated the use of chemical consolidation, in the surrounding area, of a similar nature to that provided for the outer emergency sill. It successfully retained the gravel and water and enabled the concreting to be carried out without incident. The sheet-piling was cut off at formation level before the concrete was placed.

Having secured the foundations and provided the material for the inner and outer sills, the cutting-back of the wall, provision of shuttering, and concreting of the emergency hollow quoins were undertaken. The shuttering was of timber built around a selected steel sheet-pile. When completed these hollow quoins were filled with hardwood timber to prevent damage.

The meeting face of the emergency sill was provided with an 8-in. × 8-in. × 1-in. rolled steel angle specially prepared and trued, and secured into the concrete backing by means of welded lugs at 3-ft centres. Both the inner and outer emergency sills were provided with vent filters which penetrated to the ballast. These vents were formed of three sections of pipe. The lower section is of 24 in. dia. and filled with $\frac{3}{4}$-in. selected gravel; the middle section is of 15 in. dia. filled with $1\frac{1}{2}$-in. selected gravel; and the upper section is of 9 in. dia., unfilled. A $21\frac{1}{2}$-in.-dia. concrete cap is provided with a spigot to fit into the 9-in. pipe. Five vent filters were provided at the outer sill and three at the inner sill.

No pintles or top anchorages are provided and the gates when in position would rest directly on the platform of the sill; wire strainers would be required to retain the gates.

Culverts

Three culverts were positioned in the floor of the lock on the dockside of the inner gate. Two of them contained high-pressure sewerage mains for the London County Council and were circular in section with an internal diameter of 4 ft. They were constructed of four brick-arched rings contained in the main brickwork of the floor of the lock. (See Fig. 17, between pp. 154 and 155.)

One culvert contained a 24-in. cast-iron main and the other contained two 14-in. cast-iron mains. The small size of these culverts caused great difficulty in attending to maintenance work (Fig. 9a, Plate 2). The third culvert, which contained a 12-in. gas main, one 5-in. and one 6-in. hydraulic main, and numerous electric cables, was 4 ft wide and 5 ft 6 in. high, and was likewise limited by these dimensions for maintenance attendance to the services. It was accordingly decided to rebuild all three culverts to larger dimensions within the existing floor of the lock, and it was found possible to accommodate three 6-ft-square culverts in reinforced concrete (Fig. 9b, Plate 2).

The situation of the culverts immediately east of the toe of the dam required particular consideration. In addition, the necessary excavation was below the level of the ground-water-lowering system. Consideration was given to the treatment of this area by chemical consolidation, but it was finally decided that a small portion of the original foundation would be retained and that the excavation would be carried out in a number of sections. It was unlikely that a large quantity of water would be present, and very limited pumping would suffice. This was achieved, although it was necessary to penetrate into the original ballast to relieve the pressure on the floor of the new excavation.

The pipes were carried up the wall of the excavation, and the standing head was measured. This was found to conform very closely to the records of the ground-water-lowering system, indicating that the water plane was between 6 and 18 in. below the invert of the lock. One portable 3-in. pump was found to be adequate to deal with the necessary drainage.

The construction of these culverts was carried out in fifteen sections, each culvert being dealt with separately. The centre section was first removed by excavating from the surface down to the required depth. This section was then concreted to the new dimensions, the reinforcement was left projecting, and each joint contained a P.V.C. "Expandite" sealing strip. These sealing strips were vulcanized together, forming a continuous water seal. The adjacent sections were dealt with next, leaving two end sections of the original work to complete. By this means any pressure which existed in the original construction and resulting from the toe of the dam was transmitted to the solid work.

The size of the original access shafts to these culverts were considered inadequate for satisfactory maintenance. Four shafts giving access to the London County Council culverts were combined together to form a new shaft 7 ft 11 in. internal diameter, using cast-iron segments. The new reinforced concrete culverts were brought together at the foot of the lock wall to a common sump at the foot of the new vertical shafts. It was necessary to excavate from the top of the wall to insert these cast-iron segments, and for this purpose the original access shafts were partially filled with concrete to restore weight and to increase the material remaining in the

original lock wall. The excavation for the shaft was carried out without incident, and the cast-iron segments were installed from the base and the lower three rings were pressure-grouted. The remaining work was brought up in 4-ft lifts.

The Port of London Authority Culvert had one access shaft only on each side, and in view of the future reduction of the pipework in this culvert and the rearrangement of pipes, the dimensions of these shafts were considered sufficient and they were reconditioned.

To permit the reconstruction of the culverts it was necessary to divert certain of the services. These diversions were in themselves of considerable magnitude and were carried out on the blockwork dam. The London County Council sewerage was taken in one 24-in. and one 21-in. cast-iron mains on the top course of the blockwork. The hydraulic main was diverted on the step of the second course, together with the electricity cables of the London Electricity Board. These diversions had to be completed and in operation before the original pipes and mains in the culvert could be removed. The diversions and the reconstruction of the culverts together occupied 10 months.

Concrete

In general, five grades of concrete were used:—

Grade A—Where weight only is important, e.g., in filling to cylinders, rough haunching, and heavy mass work.

Grade B—Heavy mass work, surface beds and sub-grade to pavings and roads, bedding and haunching to drains, light walls, etc.

Grade C—Mass foundations, load bearing walls, etc.
This quality was also used with a granite aggregate and classed as Cg.

Grade D—Reinforced concrete in heavy sections, including slabs, beams and columns, and for surface beds subject to heavy usage.
This quality was also used with a granite aggregate and classed as Dg.

Grade E—Reinforced concrete in light sections where great strength is required, precast concrete sections, etc.
This quality was also used with a granite aggregate and classed as Eg.

The concrete throughout was of good quality and no difficulty was obtained in achieving the specified minimum strength above.

Gates

Three pairs of new mitre gates (Fig. 10, Plate 2) have been fitted. These gates are of steel, are welded throughout and have approximately 10% reserve of buoyancy under fully drowned conditions. Each gate has a chord length of 44 ft between heel and mitre posts, with a maximum moulded depth of 6 ft and a total height overall decks of 39 ft 9 in. There are fourteen horizontal and eight vertical stiffening frames with a central division plate, and a horizontal watertight deck at frame G.

Each gate is provided with two eyeplates near the base and four lifting lugs at the top of the gate to facilitate handling by a floating crane.

Details of heelpost and mitre post are shown in Fig. 11.

The top and bottom gudgeons are of cast steel, and are bolted and spigoted to the steel gate (Fig. 5, Plate 1). The internal heads of the bolts are welded to the top and bottom deck plate. The bottom gudgeon is fitted with a phosphor-bronze bearing pad of 2 in. minimum thickness machined to a push fit and secured with two $\frac{3}{4}$-in.-dia. phosphor-bronze set-screws. The bottom pintle is of cast steel with

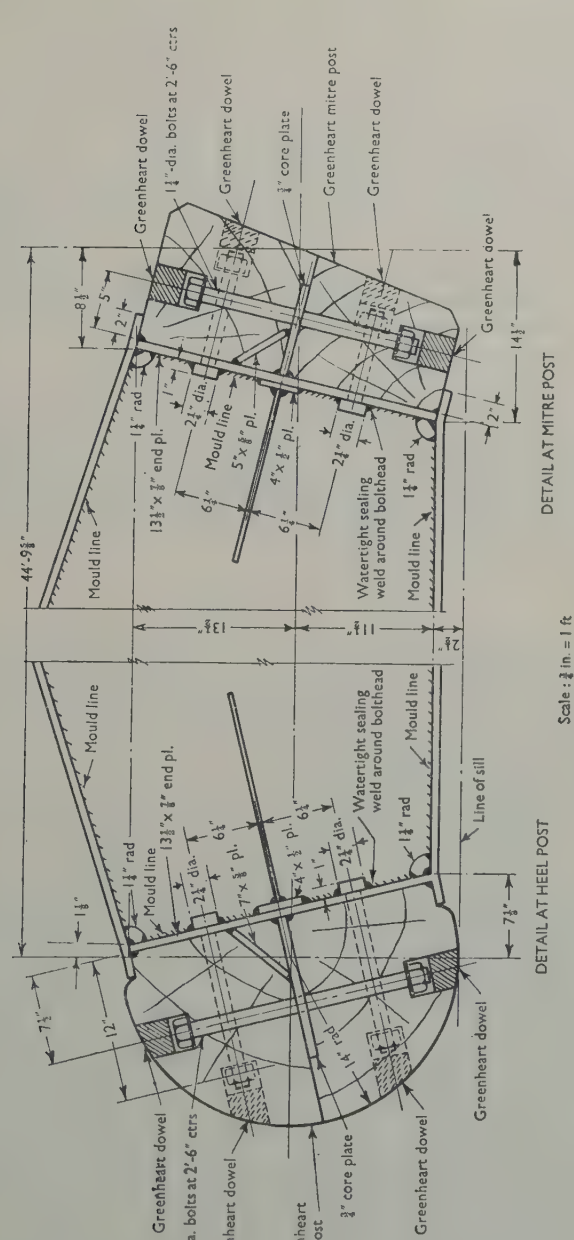


FIG. 11.—DETAILS OF HEEL POST AND MITRE POST

a radius of 17 in. against the 18-in. radius of the bearing pad. The top gudgeon has a plain bearing with a phosphor-bronze bush approximately $\frac{5}{8}$ in. thick and surrounded with a cast-steel anchor head and a cast-iron dust cap.

The anchor head and anchorage are of a conventional design with two 5-in.-dia. steel anchor-bolts set at an angle of 94°. Adjustment for alignment is given by double steel links, and for length by a forged steel coupling.

A stop fender is provided on the curved face of the gate to bear against the wall of the recess, and 12-in. × 4-in. oak sheathing on 9-in. × 5-in. bearers is provided on the flat side of the gate as a protection against barges and shipping passing through the lock.

Each gate is provided with a walkway of 4-in. timber decking on steel angle framing bolted to angle cleats welded to the top deck. Hinged steel stanchions with steel chain form a protective fencing on both sides of gangway, and are arranged to collapse to a horizontal position when the gate is in the recess.

The gates were built on the site in the lock (Fig. 18), all the material being supplied bent and fabricated. The plates were assembled, tack welded, and trued up to exact dimensions and plumb before the final welding was done. The welding was carried out by normal equipment using mild-steel electrodes. By means of a carefully arranged programme of operations practically all overhead welding was avoided.

The complete construction of the gates was supervised and inspected by Lloyds Surveyors, who also proved the watertightness.

Gate-operating machinery

Owing to the large size of the gates it was found that the width of pinion and rack of the conventional drive was so excessive that the designers were faced with the alternative of having one of the pinion bearings in the impounding water or protruding above the cope level, and since both were unacceptable the present machine was developed. (See Fig. 19, facing p. 155.)

The machine consists of a beam formed by two racks fixed back-to-back and driven by means of two pinions which reduces the rack width by half. The rack is guided at one end by means of a crosshead running between two guides, and the other end is supported on a small trod wheel running on a steel guide rail. The rack itself passes between the driving pinions, which in turn are operated by two sets of gears running in parallel and driven by one motor. The crosshead is connected to the gate by means of a connecting rod. The driving motor is fitted with a double extension shaft, each end fitted with a "Fluidrive" traction type coupling set to transmit 150% full load to each set of gears.

The two couplings are pre-set by adjusting the level of the oil to deliver a determined output. Being fluid-operated, they will act as a differential and thus keep a balance of power to each set of gears and at the same time act as a safeguard to the machinery should any obstruction occur to the gate. The fitting of Fluidrive couplings has enabled the motor to be started direct on line, and simplifies the control system for operating the machines. The gates are controlled by means of three push-buttons "open," "close," and "stop." The Fluidrives, being pre-set, act as torque restrictors and prevent overloading of the gears.

Three solenoid brakes are fitted to each machine; one brakes the motor speed and the other two are required to prevent the gate machinery from rotating through the fluid-drive when the motor is not in operation.

Built into the third-motion gearwheels are bronze-faced slipping wheels which

are intended to act as a final safeguard to the gears if, when the machine is stopped and the brakes applied, the gates should receive a blow from a ship. To smooth out hunting due to rough water a spring-loaded buffer is built into the crosshead. Limit switches cut off power to the motor at both ends of the crosshead travel whilst the usual overload trips are fitted to the motor starter.

In the unlikely event of all the safety devices failing and the machinery becoming, jammed without the operator being aware of the fact, the Fluidrives would slip causing the oil in them to overheat; this in turn would cause the fusible plugs to melt and allow the oil to escape, thus freeing the motor from the machinery and relieving the gears.

The racks are designed for a working thrust of $12\frac{1}{2}$ tons each, i.e., 25 tons applied to the gate. In normal conditions this will not be reached.

The designed time of operation for the gate is $1\frac{1}{2}$ min.

Sluices and sluice machinery

The original sluiceways at the inner, middle, and outer gates were found to be generally in excellent condition. These sluices were 6 ft. wide and had a maximum height of 9 ft 6 in. They were faced with gault brickwork and had brick arched roofs. The sluices were in granite with timber paddles, and the condition of the granite work was better than expected. Very few stones had to be renewed and it was found possible to remove all the effects of wear by dressing back 2 in. on the meeting face.

The original paddles were 12 ft high by 6 ft 11 in. wide by 12 in. thick. The opportunity was taken to increase the bearing area at the sides and bottom of the paddles, and in consequence the new paddles were of slightly larger dimensions.

The hydraulic rams were found to be in sound condition and were reconditioned and fitted with new pistons and glands for use with oil. New cast-steel slides were provided together with cast-steel crossheads. The original fixings for the sluices were removed and new rolled steel joists installed to suit the altered position of the paddles. The brickwork facing of the sluice chamber had to be cut back and the refacing was carried out in concrete. The new paddles were provided with Sherardized steel bolts and cast-steel headstock.

Each sluice is now provided with an independent motor-driven oil pump, which is situated in the adjacent machinery house. These oil pumps are of the "Deri Sine" type, having a capacity of 24 g.p.m. at a speed of 1,450 r.p.m., and are suitable for a working pressure of 900 lb/sq. in. The pumps are direct-coupled by a flexible coupling to a 20-h.p. totally enclosed fan-cooled squirrel-cage induction motor with continuous rating. The whole unit is mounted on a 100-gal mild-steel tank. The control is arranged by a solenoid-operated pilot valve. Limit trips are provided.

The actual operation of the sluice is undertaken by push-buttons with "raise," "lower," and "stop" positions. These push buttons are interlocked with the gate machines so that they cannot be effective until two pairs of gates are mitred. This safeguard is arranged to prevent the operators opening the sluices whilst the gates are closing, and thereby causing undue risk of damage to the gate. The mounting of these push buttons has been arranged in a common panel in conjunction with the gate machinery. The position of the sluice is indicated to the operator by means of a rising tell-tale.

The sluices were arranged to lower in $2\frac{1}{2}$ min at a speed of 54 in/min, and to raise in 2 min at a speed of 64 in/min.

Sluiceways

During the operations of the lock between 1887 and 1951, i.e., over a period of 64 years, a scour-hole had formed at the river end of the lock. This scour-hole, about 90 ft in diameter and up to 15 ft in depth, had been caused by the action of the water on leaving the outer sluiceways. The scour at the end of the lock apron was up to 8 ft in depth, and penetrated for a short distance underneath the apron.

The thickness of the apron was shown on the original drawings to be 7 ft, consisting of 4 ft of brickwork upon 3 ft of concrete, founded on the gravel. Core borings were made in the apron which proved that the apron consisted only of 2 ft of brickwork upon 3 ft of concrete, with indication of some settlement and undermining.

Before embarking upon extensive protective measures to prevent this recurrence, recourse was had to model experiments in an endeavour to ascertain the nature of the discharge from the old sluiceways. A 1/20th-scale model with an ample area at the riverward end, embraced the lock to a sufficient extent to give a smooth water feed. The main lock was built in concrete, with the sluices and sluiceways built of timber and made removable for alterations. A 6-in. pump maintained a continuous flow of water.

It was found that the natural conditions existing at the beginning of the contract could be accurately reproduced by a continuous operation of the model. Modifications in the sluiceways were made and tests operated to achieve a uniform flow from

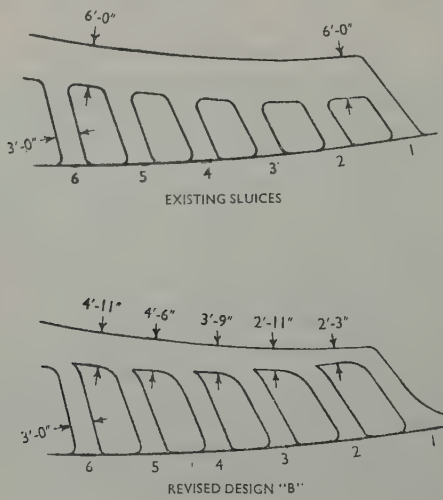


Fig. 12.—Sluices

the sluice outlets, which produced a very considerable reduction in the scouring effect. The resulting recommendations have been incorporated in the actual sluiceways (Fig. 12) at a very reasonable cost, and should prevent any further extensive scour in the years to come.

The model (Fig. 20) was built to the recommendation of Sir Claude Inglis, C.I.E., M.A.I., M.I.C.E., F.R.S., and was operated by the skilled staff of the Thames Model Investigation, who were loaned for the purpose. The construction of the model and the whole of the investigation was carried out in 6 weeks.

FIG. 13.—GENERAL VIEW OF SITE, SHOWING BLOCKYARD AND CONTRACTOR'S EQUIPMENT

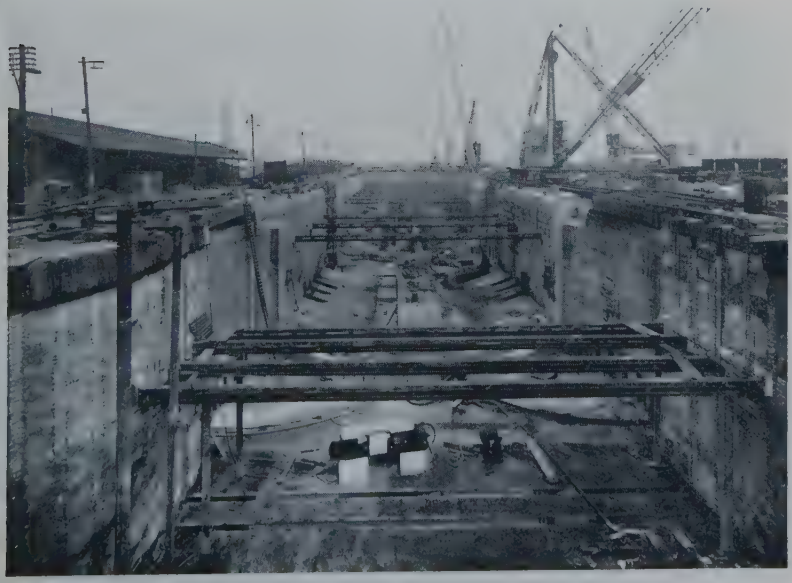


FIG. 14.—VIEW OF LOCK FROM INNER DAM BEFORE RECONSTRUCTION

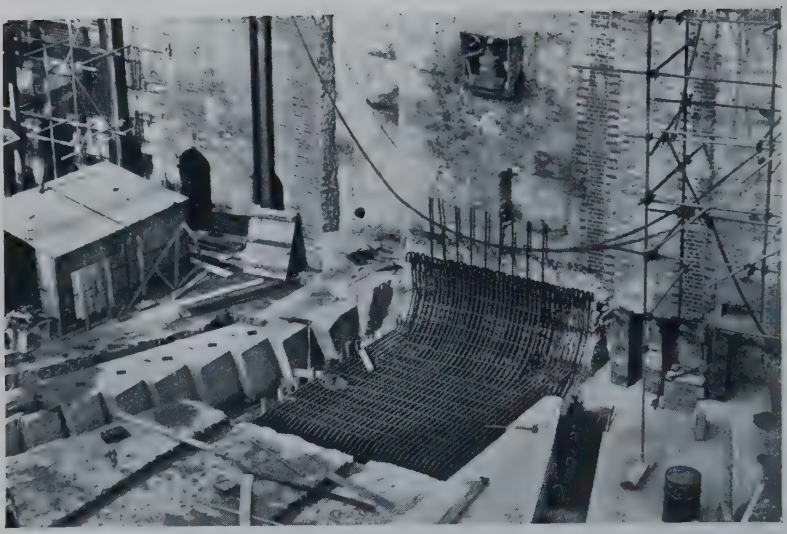


FIG. 15. —OUTER SILL, SHOWING REINFORCEMENT FOR KNEES, REDRESSED GRANITE WORK AT SILL AND OUTER EMERGENCY SILL

FIG. 16.—ORIGINAL INNER GATE SILL, SHOWING CURVED GRANITE WORK PREPARED FOR REMOVAL

FIG. 17.—EXCAVATION AND REINFORCEMENT FOR THE P.L.A. CULVERT, SHOWING ORIGINAL BRICK CULVERT

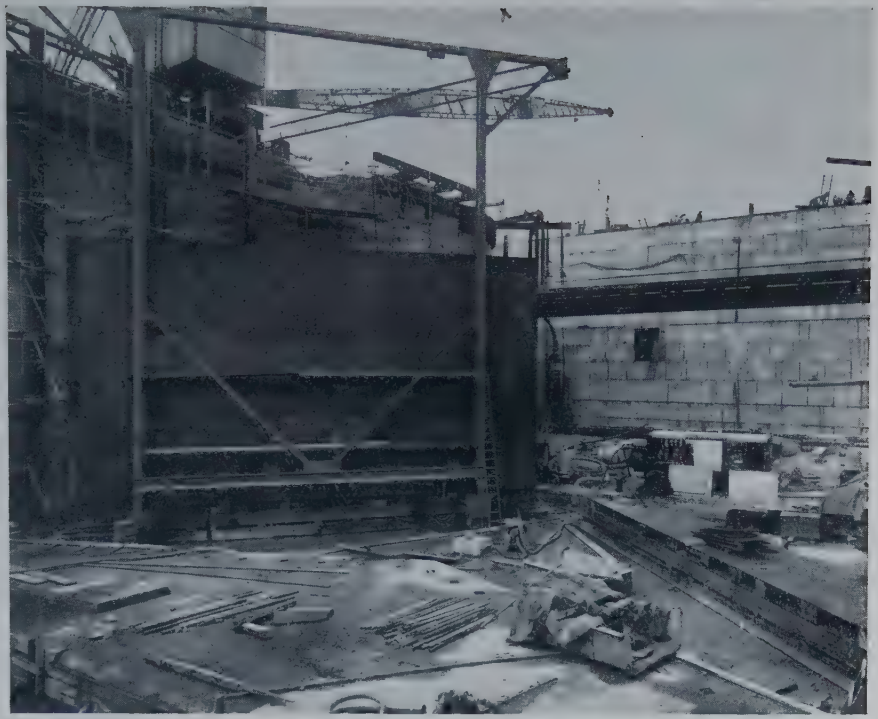


FIG. 18.—INNER GATE UNDER CONSTRUCTION. BASE FOR CONSTRUCTION OF SECOND GATE IS IN RIGHT FOREGROUND. AT THE BACK CAN BE SEEN PART OF THE INNER BLOCK-WORK DAM

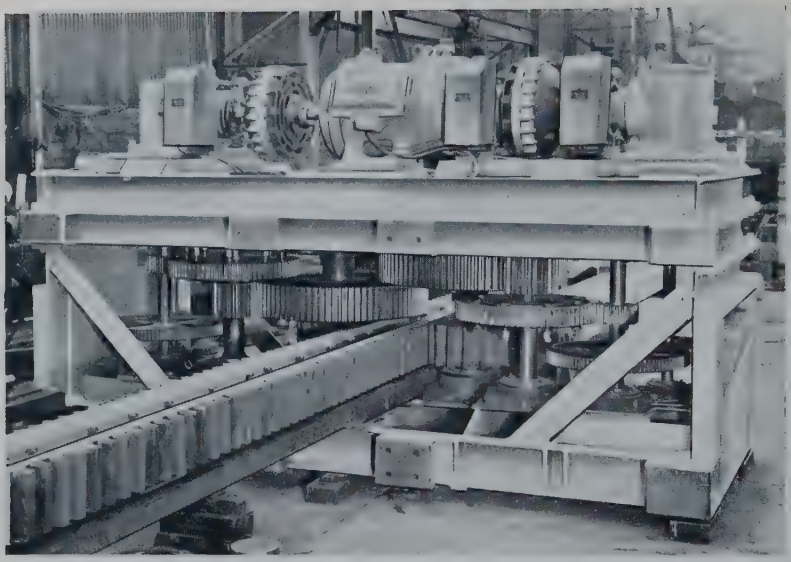


FIG. 19.—GATE-OPERATING MACHINERY ASSEMBLED AT WORKS

FIG. 20.—MODEL FOR SLUICE INVESTIGATION

Equipment

The whole of the equipment at the entrance is now electrically operated. This has enabled a complete reorganization to be carried out of the cabling and electrical supply, and to avoid the installation of a number of small controlled pillars the opportunity has been taken to house as much equipment as possible in those buildings necessary for the gate machinery. It has been possible to achieve a reasonably balanced arrangement of buildings and equipment, and to provide all the necessary requirements for the Operating Department.

Six capstans have been installed. They are electrically driven through a fluid-drive coupling at a constant speed. The barrel is of the double-drum type, having diameters 21 and 36 in. The 21-in.-dia. portion of the capstan barrel gives a hauling speed of 75 ft/min, and the 36-in.-dia. portion gives a hauling speed of 135 ft/min. Five of the capstans have a pull of 5 tons (40-h.p. motors) and one a pull of 10 tons (80-h.p. motor). The equipment is completely housed in a cast-iron box sunk below ground level. Two pedal-type foot-switches are provided for operation, one for clockwise direction and the other anti-clockwise. The control gear and the starter are accommodated in the adjacent machinery house.

Both horizontal and inclined fair leads have been provided. They are spherically mounted giving an angle of tilt up to 5°. The inclined fair lead is at 20° to the horizontal.

The bollards were retained from the original entrance. They are of the circular post type, about 60 ft apart, and are positioned 15 ft from the coping.

Two rope stores have been provided, one at each side of the lock. To reduce the number of buildings required the rope stores were extended to accommodate electrical distribution and switch rooms, and public conveniences.

A new Assistant Dockmaster's building has been erected with complete accommodation for the Assistant Dockmaster and his deputy, foreman, and lockmen; there are locker rooms and amenities for messing.

A mechanical tide gauge has been constructed on the south side of the lock to record the river level. It has a direct-reading drum which is illuminated and readily visible. At the same time an electronic repeater is incorporated to operate with the automatic recorder situated in the impounding station at the north side of the lock. The recorder is in turn interlocked with the new impounding pumps and governs the starting of the equipment.

Sewerage works

It was necessary to site the Assistant Dockmaster's office on the island between the upper and lower entrances. Provisions for sewage disposal have been necessary. Particular attention was given to this subject since the requirements are of a very moderate nature, and the dry-weather flow is only 600 g.p.d.

After consideration had been given to the provision of a small sewerage works, it was finally decided to pump the sewage under the lock and into a main on the north side. This enabled the collection tank to be of a very small capacity, but required a new 3-in.-dia. rising main throughout. The output of the installation is 90 g.p.m. and facilities have been provided for flushing the whole of the system from the lock.

Jetties

The original timber jetty on the north side has been reconditioned in hardwood timber, using the original piles. A length of 90 ft of the shoreward end adjacent

to the lock wall is of a solid construction, utilizing the concrete blocks from the invert kentledge.

On the south side of the entrance the original bullnose had been specially strengthened to receive vessels, and to enable them to turn on the bullnose for approach to the entrance. The bullnose has been demolished and an alternative method adopted using a flexible fender, 120 ft long, formed with high-tensile-steel piling at 3-ft centres, surmounted by a continuous reinforced concrete capping beam. These works are not yet completed.

Electrical

A diagram (not published) which accompanies the Paper shows general electrical arrangement for the entrance.

The provision of a new high-tension intake and the conversion of this area to 3-phase A.C. supply have enabled a very satisfactory arrangement of electrical cabling to be undertaken. Safeguards have also been incorporated to prevent the possibility of sluices being opened until the gates are closed. In addition, duplicate control has been provided at the south side of the lock from which a pair of gate machines can be controlled simultaneously.

New lighting standards, at about 85-ft centres and 25 ft from the coping, are provided with special dispersive reflecting lanterns.

Acknowledgements

The Paper is presented by permission of Mr G. A. Wilson, M.Eng., M.I.C.E., M.I.Mech.E., Chief Engineer, Port of London Authority, to whose designs the work described was undertaken.

The principal contractors were Messrs John Mowlem & Co. Ltd, with Soil Mechanics Ltd as sub-contractors for the ground-water-lowering system, and Messrs Head Wrightson & Co. Ltd, sub-contractors for the gates and machinery.

The Paper, which was received on the 30th September, is accompanied by eight photographs and fourteen sheets of drawings, from some of which the half-tone page plates, folding Plates 1 and 2, and the Figures in the text have been prepared.

Discussion

The Author introduced the Paper with the aid of a film.

Mr J. R. Malcolm (Senior Engineer and Agent, John Mowlem & Co. Ltd) observed that from the contractor's point of view there had been many difficulties and problems to solve, and amongst them were the construction of the two dams and the best method of constructing the knees. The dams had required careful consideration and especially the outer dam, where, owing to the presence of a scour hole about 20 ft deep in the chalk, it had been difficult to provide a satisfactory foundation for the blockwork dam originally proposed. After some consideration it was decided to abandon the idea of a blockwork dam and to use a steel sheet-pile dam, as shown in the Paper. Both the dams had proved to be remarkably tight; in the case of the outer dam practically no pumping was necessary at low water, and at high water an 8-in. pump was able to deal with the flow. At the inner dam a 6-in. pump was adequate.

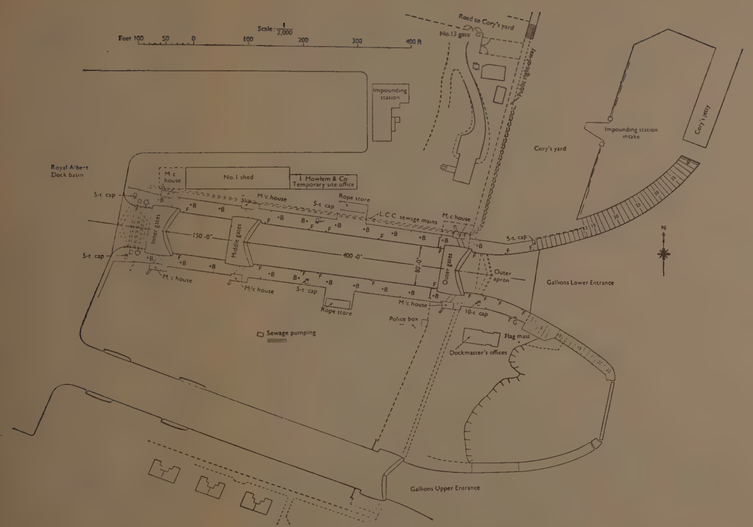


FIG. 1.—SITE PLAN

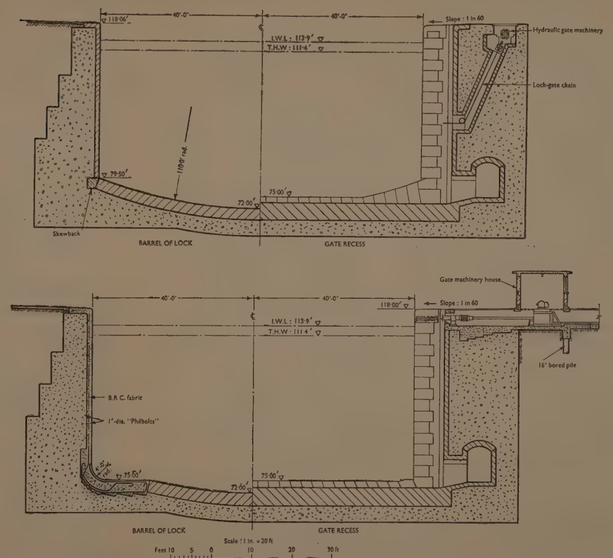


FIG. 2a.—CROSS-SECTIONS THROUGH LOCK BEFORE AND AFTER RECONSTRUCTION

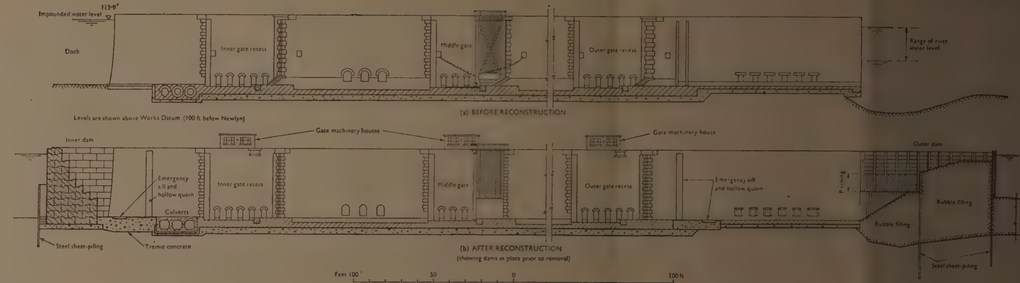


FIG. 2b.—LONGITUDINAL SECTIONS THROUGH LOCK BEFORE AND AFTER RECONSTRUCTION

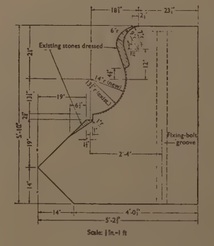


FIG. 3.—DRESSING OF EXISTING GRANITE QUOIN STONES

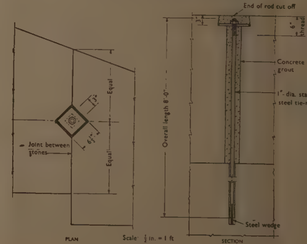


FIG. 4.—ANCHOR FOR GRANITE STONES

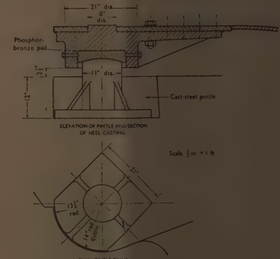


FIG. 5.—PINTLE DETAIL

J. A. FISHER

The Institution of Civil Engineers. Proceedings, Part II, June 1956

WILLIAM CLOWES & SONS, LIMITED: LONDON

RECONSTRUCTION OF THE GALLIONS LOWER ENTRANCE LOCK AT THE ROYAL DOCKS OF THE PORT OF LONDON AUTHORITY

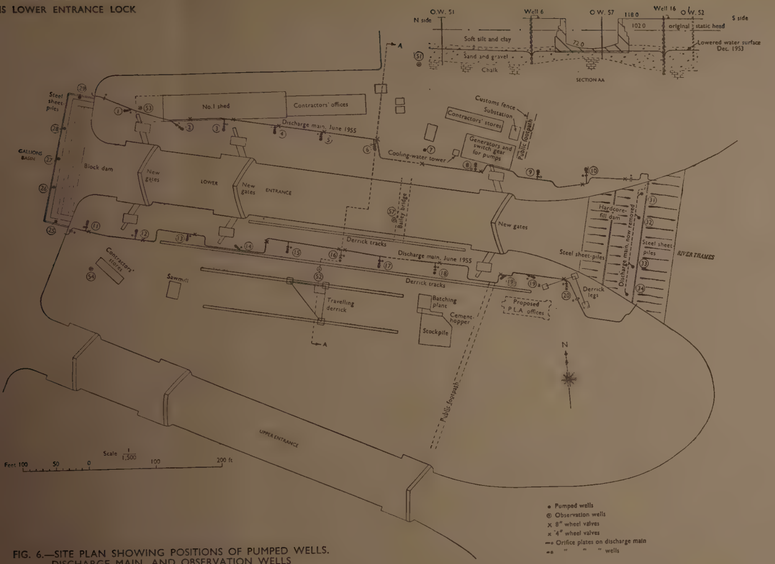


FIG. 6.—SITE PLAN SHOWING POSITIONS OF PUMPED WELLS, DISCHARGE MAIN, AND OBSERVATION WELLS

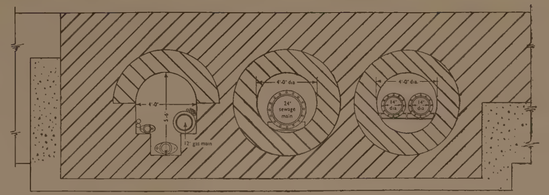


FIG. 9a.—CROSS-SECTION OF ORGINAL CULVERTS

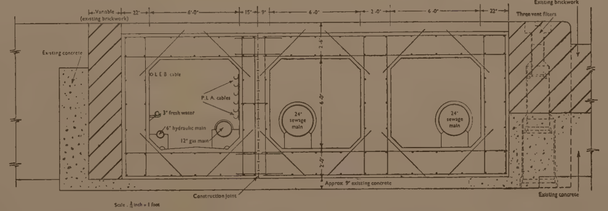
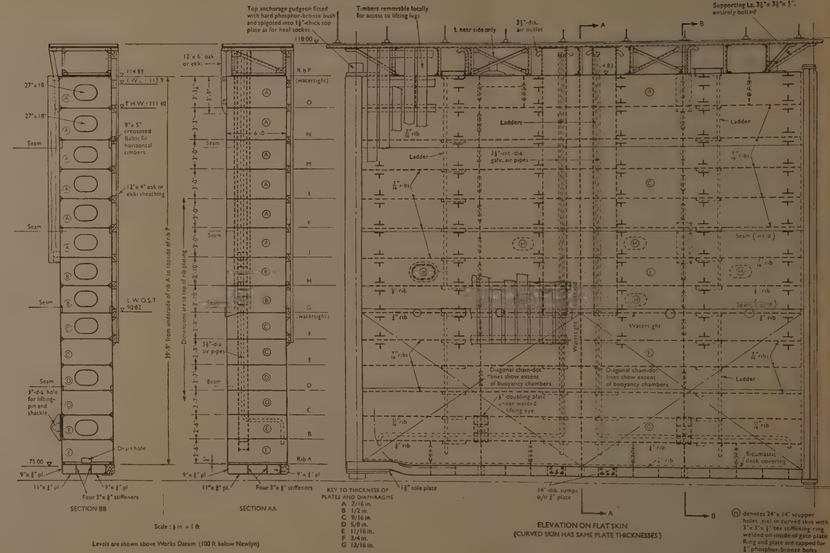


FIG. 9b.—CROSS-SECTION OF NEW CULVERTS


Top anchorage gudgeon fitted with hard phosphor-bronze bush and spigoted into 1¾"-thick cap plate as for heel socket
Timbers removable locally for access to lifting lugs
L near side only
3½"-dia. air outlet
Supporting La. 3½"w 3½"w 5/16". entirely bolted
A
B
11'-8 3/4"
12'-6" oak or ekki
R & P (watertight)
O
N
M
L
K
J
H
G (watertight)
F
E
D
C
B
R & P
27'-1 1/8"
27'-1 1/8"
Seam
Seam
Seam

To construct the reinforced concrete knees at the foot of the existing walls it had been necessary to make pockets in the invert 11 ft wide at 55-ft centres. The sides had to be cut plumb and true to dimension to allow the reinforcement to be placed accurately so as to give the correct spacing between the bars in adjacent sections and also to allow the concrete to be poured for the whole 11-ft width of the section, so as to avoid the use of shuttering or excessive trimming of the concrete when the adjacent section was excavated. After some trial it had been found that to cut a chase round the three sides of the pocket gave the best results, using a Siskol mining cutter. Another advantage of using the Siskol cutter in that way was that when blasting took place there was a free face on three sides of the area, which stopped the shock going to the rest of the work.

Mr Malcolm illustrated his remarks with a series of lantern slides. The first showed one of the pockets and the straight face obtained by using the Siskol cutter, also the various steps which had to be cut in the bottom, which required careful work.

The Siskol cutter was secured to a 10-ton kentledge block, which provided a steady base and enabled an accurate cut to be made. The block was handled into position by a 15-ton derrick. The cutter could be swung in a vertical plane about an axis and at the same time raised and lowered to control the depth of cut. Various lengths of boring bar were available depending on the depth of cut to be made. The cutter head which was most suitable had five chisel-shaped prongs. The cuttings were removed by means of a rake.

Timber shuttering was used for the curved parts of the knees. It was handled as a unit, placed on previously levelled packings and weighted down with eight 2-ton kentledge blocks.

The quality of the concrete in the old walls varied considerably and in some places a good deal of it had to be cut out. Pockets were cut in the face of the walls to give a key for the reinforced concrete facing, in which B.R.C. fabric was used. The shuttering used was of timber faced with plywood in panels 5 ft 6 in. long and 3 ft 6 in. high, and a large number of uses was obtained from those without trouble.

At each end of the lock emergency sills and hollow quoins were provided and the latter were formed in the concrete facing of the wall. To make them as true as possible without dressing after the concrete was placed a quoin shutter was made in one length to the full height of the wall. It was made of timber faced with plywood and assembled on a No. 5 Larssen pile. The shutter was accurately located at the bottom and bolted back to the wall by 1¼-in.-dia. bolts. At each side of the quoin shutter climbing shutters were arranged which brought up the lifts of concrete as required.

As mentioned in the Paper, most of the granite work in the lower part was quite good. There had been an attempt to repair one of the old sills under water by cutting back the granite, inserting steel bolts, and securing a reinforced concrete patch, but the patch had not held.

The gates and sluices were tested with a head of 40 ft of water on the pressure side only, and all the gates and sluices were satisfactory at the first test.

The work had normally to be carried on until 6.15 each evening, so that during the winter lighting was necessary. A system of sodium lighting had been provided, which had given excellent results.

Mr F. H. Allen (Assistant Director, Hydraulics Research Station, D.S.I.R.) remarked that he had been concerned with the model investigation (described briefly on p. 154), which had been undertaken to find out whether or not the quite considerable scour which was known to have taken place on the riverward side of the apron could be reduced by relatively small alterations to the sluice outlets. The experiments had been carried out by Mr W. A. Price and the Port of London Authority's model research staff, under the direction of the Hydraulics Research Station of D.S.I.R.

Mr Allen showed several slides to illustrate his remarks. The model was constructed to a scale of 1:20 vertically and horizontally. The bed of the riverward section had been moulded in sand which had a mean diameter of about 0·2 mm. The results obtained from tests with one design of sluice had been compared with those obtained from tests on various

revised designs; the relative merits of the designs had thus been compared, but no attempt had been made to draw any conclusions about the actual amount of scour. Since the model had been constructed in 3 weeks and the experiments carried out in 6 weeks there had been relatively little time for an academic research investigation. The model had been operated under the most severe conditions, i.e., the conditions liable to cause the greatest scour. That had been done by maintaining high-water impounded dock level in the lock and L.W.O.S.T. level in the river, and allowing the water to discharge continuously under those conditions through the sluices. The water was circulated in the model by means of a system of pumps, pipes, and constant-head weirs.

Each main sluice was originally of constant width throughout, and the outlets or sluiceways branched sideways from it. When that old design was tested in the model, it had been found that by far the greatest part of the discharge passed through the final or riverward sluiceway; and the Author's film, taken when the lock had been filled again after the completion of the works, showed that that was borne out in nature. A small amount of water discharged through the next sluiceway and very little through the others.

The aim had been, therefore, to equalize the flows from the sluiceways and thus to equalize and reduce the jet velocities, so avoiding the highly turbulent conditions which were the outcome of the original design. That had been done in the first place by reducing the width of the main sluice in five roughly equal steps; at the same time the entrance to each of the sluiceways from the sluice was formed as a slight scoop, so that part of the water was encouraged to flow into each sluiceway. In addition the outlets were flared. That produced very encouraging results, but it transpired that there was some difficulty in reproducing the flared outlets on the actual structure because of the reinforcement of the concrete. A revised design (B) had therefore been tested in which the flared outlet was used only for the riverward sluiceway and the others were left unaltered. The reduction in the width of the sluice was as in the first revised design (A). The third revised design (C) was an attempt to equalize the discharges still further by slightly altering the reductions in width of the main sluice, but that had not been altogether successful.

The model was run for 9½ hours with the existing design of sluice, and the scour hole which developed was very similar in depth and shape to that formed in nature. With design A two scour holes developed. The maximum depth of scour, measured after an experiment of the same duration, was relatively small, being in the region of 8 ft as against 18½ ft. That design had not been adopted, partly for structural reasons, and partly because there was some slight disturbance and scour at the edge, which it was thought might possibly cause some damage to the wing walls. Design B showed a return to the development of one scour hole, with a maximum depth of only 10 ft, as against 18½ ft, and no scour at the edges. Design C was similar, but the depth of the scour was a little greater. Design B had therefore been recommended and adopted.

Mr G. E. Wild remarked that the lowering of the ground water had been an important feature of the works. The system started working in July 1953. There had been a short pause while the abutments of the outer dam had been made watertight and then it restarted in September 1953, and ran continuously until Christmas Eve, 1955, i.e., about 2¼ years. During that time it had been pumping water at the rate of about 2,000 g.p.m. It started off as 2,500 g.p.m. but, as expected, the discharge decreased in the first 2 or 3 months and thereafter a discharge of about 2,000 g.p.m. had been maintained consistently. That was extremely close to the predicted amount, which was pleasantly surprising in view of the fact that in calculating the quantity to be pumped it was necessary to select a permeability for the ground on the basis of past experience, because in Thames gravel the permeability locally could vary enormously and laboratory tests for permeability on such soils were not of much avail. A good deal of prior information, however, had been available from other jobs. It has also been necessary to decide whether any allowance ought to be made for the fact that the river and the lock, sources of water at a constant head, were well within the radius of influence of the wells. The decision arrived at had been that they were probably so silted up that they would not greatly affect it, and that proved to be the case.

If it was necessary to pump 2,000 g.p.m., the question arose of whether or not twenty wells were really needed. Mr Wild showed a slide representing longitudinal sections through the two lines of deep wells on the north and south of the lock. When the matter was being considered at the tender stage, borings were available only at the east and west ends of the lock and there the chalk underlying the gravel was considerably higher. In the boring of the wells it was discovered that there was a much lower area in the middle. Had that been known at the start, a small number of very much larger sumps might have been a satisfactory alternative. That had in fact been considered in the early stages of design, but the use of sumps as opposed to deep wells had been discounted, because, if it was necessary to get the water down very close to the chalk, in order to provide a sufficiently large wetted perimeter the sumps would have to be very big, and the cost of sinking them without losing any ground at all behind the old lock walls would have been very high. It would also have been necessary to take special precautions to prevent the corrosion of the sump materials, which might have caused a subsequent collapse.

Once it had been decided to use bored wells of a size where only one pump could be installed in each well, and where in consequence the well was out of action whenever the pump had to be cleaned, it was not just a matter of dividing the anticipated quantity by the capacity of the pumps; it was necessary to allow for wells being out of action, and also for a great deal of variation between the yields of individual wells. With some of the wells when pumping was first started there were yields of 40 g.p.m. per foot of immersed depth, whereas with others the figure was less than 6 g.p.m. After about 2½ years of pumping the yields had about halved. That arose from the deposition of oxide in the filter gravel round the actual wells. Mr Wild considered that the number of wells used was just right, with the exception of the outer dam and the inner dam.

With regard to the standby arrangements for electrical supply, the Author mentioned that generators had been provided which were capable of running the whole supply, and those were started manually by operators who were on constant duty. That was possible because it was not a full artesian case. It had in fact been calculated that in the bulk of the lock it would take about 5 hours for a dangerous pressure to reassert itself on the base. At the riverward end, however, where the chalk was very high and the silt was a little lower, there was a full artesian case and it had been expected that the pressure would reassert itself very quickly. That had in fact proved to be the case when the pumps had been stopped and readings taken on the recovery of pressure. It had taken about 8 hours for the pressure to reassert itself in the centre of the lock, but at the riverward end the pressure had in fact reasserted itself within half an hour. Had it been a full artesian case, the pressure might be fully re-established in a matter of minutes. That had occurred on previous similar jobs, and there it had been necessary to have fully automatic changeover from the main supply to the standby generators.

Mr W. J. Sivewright (Assistant Engineer (Civil), Port of Bristol Authority) said that he proposed to confine his remarks to the alterations to the sluices and the question of the scour hole, with particular reference to similar work which had recently been carried out by the Port of Bristol Authority. At the 100-ft entrance to the Royal Edward Dock at Avonmouth a scour hole 100 ft across and 35 ft deep below bed level had developed since 1908. In the case of the Royal Edward entrance the apron consisted of monoliths 23 ft deep and founded 4 to 5 ft in red keuper marl. Above that there was merely soft silt. The sluice culverts were about 8 ft wide and 11 ft 3 in. high, each with two openings into the outside channel, their cross-sections being 6 ft wide and 9 ft high.

The scour had extended so far that the red marl under the monoliths was beginning to scour away, and, although the undermining of the monoliths was not so extensive as to render them unstable, the state of affairs which prevailed could not be allowed to continue indefinitely. Consequently a model, largely constructed of timber, had been made in the Authority's workshops, primarily to determine what material, if any, could be used to fill the hole and prevent further undermining, and also to investigate what other measures could be taken to minimize future scour.

The model in question was 1:50 true to scale, which suited the available room for erecting it. As in the case of the Gallions model, water was circulated by means of a pump. Only a short length of the lock above the gates had been reproduced, and a series of baffles was used to still the water before it passed through the model sluices. The pump discharged into the lock and an adjustable weir maintained the level of water behind the lock gates. The surplus water drained back to the storage tank. The model extended between the solid entrance piers for a distance of five lock widths beyond the length of the apron. The level of the water in the entrance was controlled by an adjustable weir, after which the water passed over a rectangular measuring notch before coming back to the storage tank for recirculation.

What was the rate of flow from the Port of London Authority sluices and how was the actual flow measured? The flow adopted in the Gallions model was presumably to the $2\frac{1}{2}$ power of the scale and it had been stated that the model was operated with high water inside and low water outside. In the case of the Avonmouth entrance, the discharge from the actual lock had been measured by timing the water level past the gauge marks on the lock wall with the sluices wide open, and it had been surprising to find that for equal decrements of water level the increments of time varied considerably. The plotted figures of discharge against time showed a rhythmic variation, evidently due to a surge of water up and down the lock; but when plotted for the first 15 ft of discharge the graph of discharge against time varied very little from the best parabola through the plotted points and very little from a straight line through the points. The flow was 2,200,000 g.p.m. with normal dock water level inside the gates and mean water low springs outside, giving a head of 41·5 ft. That had been taken as the basis for the model, for the same reason as in the Gallions model, that presumably most damage was done with the greatest difference of head.

As a first assumption the same coefficient of discharge was taken for the model and for the actual sluiceways, and the actual flow divided by 17,700 corresponded in the model to a 1-in. depth over the measuring notch. With the water in the model corresponding to normal dock level the depth over the notch was 1·1 in. This difference was accounted for by the difference in coefficient of discharge for the model and for the actual sluiceways. That was considered sufficiently accurate having regard to the measurements in the model and lock. Qualitative observations on the character of flows from the actual lock and in the model were made and confirmed the general accuracy of the model.

The bed material outside the model gates was estuarial sand with a grading of 70% passing No. 7 sieve and 1% passing No. 52, with various intermediate gradings. The model started with a bed of sand at a level equivalent to the general level of the bed of the entrance and was run until equilibrium was reached, in about half an hour. The scour hole then closely resembled the actual one and the general topography of the entrance bed was closely reproduced. There was a tendency for the coarser material to be deposited on the outside edge of the scour hole. That was removed by hand, which was perhaps cheating, but this procedure was considered justified since if the material had been finer it would have been transported farther down the model. Model concrete cubes representing cubes from 1-ft to 7-ft side were all swept away from the apron with the sluices running full open.

A long series of experiments had been carried out to examine the effect of filling the hole with rough stones of various sizes and with sand. Other experiments had been done with various water levels in the entrance and also with Rehbok teeth on the edge of the apron, but those had no effect, since the maximum size of the teeth was limited to the 2 ft difference between the apron and sill and was so small as to be ineffective.

Mr Sivewright noted that the alteration of the culverts in the Gallions entrance improved the flow. Had those alterations altered the total discharge from the sluices and altered the normal time for locking? At Avonmouth, as at Gallions, most of the water discharged from the downstream outlet, and both in the actual lock and in the model water entered the upstream outlet at the commencement of sluicing. Experiments had been carried out using diverters of various shapes, but without appreciable effect. The alteration of the main sluiceway, as had been done by the Port of London Authority, had not been

considered, nor was it practicable. It was worth observing that the inlet end of the Avonmouth sluiceway culvert approximated in shape to the outlet end of the Gallions culvert as reconstructed.

What was the nature of the bed beyond the end of the apron at the reconstructed Gallions entrance? Presumably the base of the rubble dam had been left in position.

The main conclusion from the Avonmouth experiment was that the velocity of discharge should be reduced. That might seem self-evident, but the question of locking time came in. The reduction of velocity achieved by permanently throttling the sluices to three-quarters open, thereby reducing the velocity to 50%, had made relatively little difference to the locking time. It was also concluded that 2-ft-cube stones in the hole would stay in place if the level of the stone was sufficiently below the top of the apron. There was no point in filling the hole since the Port Authority was concerned only with protecting the monolith bases.

Mr Sivewright concluded his remarks by showing several slides of the model.

Mr A. L. Reed (Technical Manager, Dock & Harbour Equipment Dept, Head, Wrightson & Co. Ltd) commented on the Author's statement that the gates were fully assembled at site in the lock bottom, all the material being supplied shaped and fabricated from the works, and explained the method of assembly. The gates had been built in the vertical position on a timber raft placed adjacent to and in line with the quoins. The raft was in two parts, one on top of the other, separated by 9-in. × 3-in. timbers suitably spaced for the eventual placing of the rollers for rolling the gates into position. The bottom deck was placed on the raft and the diaphragms were tack-welded into position. The first strake of the flat skin plating was tack-welded into position. The decks and diaphragms were assembled and tack-welded up to the level of the first butt on the flat skin plating. The centre-lines of all plates were marked and the corresponding contact points of centre-lines of contacting plates were also marked. That was essential for correct assembly and helped considerably in checking.

The first curved skin was then brought up to the four decks and diaphragms previously assembled. That curved skin had not been preformed in the works but was pulled into shape by Yale "pull-lifts." At that stage the rigid squaring frame shown in the Author's film was brought up to the flat skin plating. That rigid frame consisted of joists and channels and served as a support to the flat skin plating during further erection. It proved exceedingly useful for checking the alignment against twist and distortion during the remainder of the assembly.

The remaining deck, diaphragms, and skins were assembled in a similar manner, except that the flat skin was assembled first and a sealing run put in before the plate was tack-welded to the diaphragms and decks. Similarly, with the curved skin the sealing run was put in before the plate was fastened to its diaphragms and decks, the reason being that there was a certain amount of preliminary shrinkage and that was a means of minimizing distortion. After all the decks and skins had been assembled the heel- and mitre-post plates were brought into position and tacked in position, tacking to the flat skin first. The gate had now been fully assembled, but was only tacked except for the sealing runs in the butt welds.

The next stage was to put sealing runs in the vertical positions on all diaphragms and sealing runs on the downhand and overhead welding inside the gate, and then complete those welds with the exception of the access trunks. Four runs were then put on the outside of the butt welds and chipped back on the inside, and the inside welding completed before going outside and finishing the butts. Next the outside welding on the heel and mitre posts was completed—one welder working on the flat side and one on the curved side. The access trunks were welded last; it had been felt that to weld them as the gate was proceeding would cause another obstruction to the natural distortion of the gate. Four welders were continuously working while the final welding was taking place. The procedure was that they started in the centre of the gate and worked out towards the heel and mitre posts, and up and down from the centre towards the top and bottom of the gate.

At all stages of assembly and while welding was in progress it was essential that all plumb lines and levels should be checked twice daily, on commencement of work in the morning and on commencement of work in the afternoon. That gave a chance of removing distortion before it was permanently welded into the gate. The steelwork was approximately $\frac{5}{16}$ in. larger than was really required, in order to allow for shrinkage during the welding of the gate.

On completion of the steelwork the greenheart had been fitted and dressed to dimensions taken from the finished standing sills and quoins. The gates were then rolled into position, as shown in the Author's film, and lowered in stages by jacks on to the pintles and finally connected up to new top anchorages and the operating machinery. The three pairs of gates had now been completed, and they had been tested with a full head on the pressure side and dry on the sill side. The inner gates had negligible leaks, but it had been decided to make the other gates approximately 0·010 in. longer than the inner gates, and on test those proved to be perfectly watertight.

With regard to the machines, only temporary electric wiring had so far been provided, but that apparently worked very well. The permanent wiring would not be connected up for about 2 months, since interlocking with the sluice machinery was necessary.

Mr J. A. Dempsey (Assistant Site Engineer, John Mowlem & Co. Ltd) said that in the past 40 years only three other dams [2, 3] had been constructed of precast concrete blocks in a manner similar to the inner dam described by the Author, and therefore certain techniques and difficulties which arose during construction were worthy of mention. The dam had been designed as a simple gravity section founded on the natural gravel, capable of withstanding a horizontal thrust of approximately 32 tons/ft run. Under the gravel, which was about 6 ft deep, was the chalk. Both those strata were adequate to support the mass of the dam, but their ability to take the horizontal thrust had to be the subject of careful investigation. Shear-box tests had been carried out on uncompacted gravel in contact with a rough concrete surface and it was found that a movement of approximately 0·8 in. was to be expected before the full frictional resistance of the gravel was mobilized. That corresponded to a factor of safety of about 1·4. That movement under the full hydrostatic pressure would seriously endanger the watertightness of a clay seal placed between the cut-off and the face of the dam, and, once that seal had been broken, there would be a serious increase in the uplift pressure under the dam. The factor of safety against sliding would then be reduced to 1·05. Neither of those two factors of safety was acceptable in practice, especially in view of the risks to the whole dock system should any failure occur. Moreover there would be a risk of piping in the gravel should seepage flow become at all large. Therefore, it had been decided to install a row of deep wells between the sheet-pile cut-off and the face of the blockwork with a view to intercepting any seepage water which might come through the cut-off and eliminating the danger of a break in the clay seal at the face of the dam.

Furthermore, the Port of London Authority proposed to extend the existing sill a further 26 ft, to within 15 ft of the toe of the dam. To carry out the construction of that sill would have meant excavating to a depth of 5 ft below the base of the dam. They proposed that the work should be carried out in the dry. Such an operation would have been difficult and might have seriously affected the stability of the dam. The contractors proposed, therefore, that the new sill should be constructed at the same time as the dam, and under water,[4] and that it should be extended right up to the toe of the dam, so that it would act as a strut between the existing sill and the toe capable of carrying the full horizontal thrust. That modification increased the factor of safety of the foundation as a whole to 7, and eliminated all risk of sliding and the problems associated with it.

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[2] P. W. Bertlin, "The Construction of the New Entrance to Tranmere Dock." Min. Proc. Instn Civ. Engrs, vol. 221, p. 241 (1925–26, Pt 1).

[3] George Ellson, "Dover Train-Ferry Dock." J. Instn Civ. Engrs, vol. 7, p. 223 (Dec. 1937).

[4] See footnote 1, p. 143.

The deep wells which he had mentioned were left in the design purely as a precaution, mainly with a view to reducing any uplift pressure under the new sill caused by the seepage water. In the event, those deep wells produced so small a yield that several were abandoned at the start.

The other point which he wished to mention was the sealing of the face of the dam. There the joints between all the blocks in the water face of the dam had a recess cast round the face of each block, and that was filled by a canvas sleeve placed by the divers under water. The canvas sleeve was then pumped full of grout until it entirely filled the recess, thereby closing the horizontal and vertical joints between the blockwork.

Once that sealing was complete the dam was tested under the water load. It was when they had approximately half the full differential head on the dam that considerable horizontal movement took place between six of the courses of blockwork, and the seal in the face of the dam was broken. That horizontal movement had been arrested only by interlocking recesses cast on the top and bottom faces of all the blocks. A survey had been carried out as soon as that movement was noticed and showed that a displacement of approximately 1 in. had occurred between adjacent courses. That corresponded roughly with the tolerances in the interlocking recesses. Up to that time it had been thought that the interlocking recesses were not entirely necessary to prevent sliding and that the frictional resistance between the concrete surfaces was sufficient; nevertheless those interlocking recesses had been included in the design of the Tranmere Block and also at the Victoria Dock, Hull.

From calculations which had been carried out after the survey it appeared that the angle of friction between the blocks must have been as low as 17½° for sliding to occur. Normally an angle of friction of between 27° and 30° might be expected for moderately smooth surfaces of concrete. The reduction was in all probability due to the presence of silt mixed with oil which was continually deposited over the site and which no amount of cleaning would entirely remove. It had a fine and greasy nature which probably caused it to act as a lubricant between the courses. The survey also showed that there was little or no danger of any further horizontal movement since the tolerance in the interlocking recesses had been taken up. As a precaution, however, a further course of blockwork and a quantity of old rails had been added to the crest of the dam, in order to increase the frictional resistance.

In the places where relative movement had taken place between the adjacent blocks it was necessary to caulk the joints by diver. When that had been completed, the lock was successfully dewatered. Screened ashes were tipped over the face of the deck at intervals during dewatering, and for a period afterwards, to control the small leakage. As Mr Malcolm had mentioned, the leakage throughout the reconstruction work was quite small and had been dealt with by a single 6-in. pump, which Mr Dempsey believed worked intermittently.

Mr Cecil Peel (Assistant Chief Engineer, Civil, Port of London Authority) said that the Gallions Lower Entrance Lock had had rather a famous career. It had formed the major lock entrance of the Royal Albert and Royal Victoria Docks for many years, from 1886 to the opening of the King George V Entrance in 1921. It had carried a vast quantity of shipping, and it had still been used for shipping up to about 1939, although its condition had become steadily worse. Had it not been for the war, its reconstruction would probably have been undertaken at an earlier date.

The work described formed one of the larger items in the large programme of works which the Port of London Authority had carried out since the war, and it was considered that it would give a new lease of life to what was a very valuable entrance. In an impounded dock system it was obviously necessary to have an entrance large enough for the largest shipping likely to use the docks, but the existence of a smaller entrance was an invaluable asset because the majority of the ships were smaller in size than the largest. Gallions Lock had been found on a dimensional analysis to be capable of taking about two-thirds of the ships using the Royal Docks, and that was sufficient to demonstrate its importance.

Unfortunately, the old shape of the sills and invert had imposed a restriction on the depth available. The lowering of the sills and invert which had been carried out, although it had been costly, had given a valuable increase in that depth. The maximum size of ship which could use the lock would be about 510 ft long by 70 ft beam, with a maximum draft of the order of 29 ft 6 in. Formerly, such vessels could be locked at the Gallions Entrance only at high-water of spring tides, and at neap tides the maximum draft had been about 3 or 4 ft less. It would now be possible to lock vessels having a draft of 29 ft 6 in. at high water of almost every tide, which was a great improvement. A further consideration which would show the importance of that was the fact that each foot of draft of the vessel meant an additional 500 to 600 tons of cargo.

There were other important considerations in using the Gallions Entrance as a relief to the King George V Entrance. In the impounded system in question all the water had to be pumped from the river and, since the amount of water used at the Gallions Entrance was only about two-thirds of that used at the King George V Entrance, the more vessels that could be locked at the former the greater the saving in pumping the impounded water. Again, the water from the river was very silty and a great deal of that silt settled in the Albert Basin and had to be dredged. That meant that the water used for locking at the King George V Entrance was relatively clean, whereas that which would be used and which formerly had been used at the Gallions Entrance was relatively dirty, so that there would be a double saving, with less silt deposited and more dirty water run back to the river.

In choosing at the design stage the methods to be adopted to dry out the lock, the most important feature to be considered in connexion with the work had been the stability of the walls and floors. The design of the dams had been left to the contractors, but the installation of means to lower the ground-water level and to load the centre of the invert had been specified. That had been done after careful consideration and calculation, which showed that whilst the margin of stability of the walls was not large, it was sufficient provided that there was no free hydrostatic pressure acting on the back of them. Furthermore, since the invert had to be cut into it was necessary to lower the water pressure underneath the floor. It would have been possible, of course, to stabilize the walls against overturning or collapse by an elaborate system of strutting, but that would have cost a great deal of money and would have been a nuisance during the re-facing, so that the advantage lay from every point of view with lowering the ground-water level. Preliminary calculations showed that they could expect to get the water level down to something in the region of + 80 works datum on the simple assumptions used. It was also considered that since the ballast in the bottom of the basin and on the bed of the river for a considerable distance from the lock was thickly covered with mud there was a good chance of obtaining an effective seal, and that the water could be taken down lower than the original calculations suggested. The actual results had been extremely satisfactory. The contractors had in fact put some struts across the lock as a precautionary measure, but during most of the time the lock was empty those struts had not been tight and had been carrying little if any pressure. The lowering of the ground-water level produced considerable settlement of the ground at the back of the walls, and no doubt that led to considerable cohesive drag on the back and so assisted in their stability.

It might be wondered why electrical machines had been installed to work the gates, instead of hydraulic. That was simply due to the fact that all the cranes at the Royal Docks were electric, and it was no longer necessary to maintain the many miles of hydraulic mains. The hydraulic system at the docks had therefore been remodelled and curtailed, and it had been felt opportune to install electric machines at the Gallions Entrance. Such machinery was commonly used on the Continent and in the United States, and he had no doubt that it would work equally well at the Gallions Entrance.

Professor A. W. Skempton (Professor of Soil Mechanics in the University of London, at Imperial College) remarked that there had been no reference in the Paper, or discussion, to the movements which took place in the outer cofferdam, and he thought they should be recorded. Together with Dr A. W. Bishop, Professor Skempton had been consulted by Mr R. Glossop, of John Mowlem & Co., concerning the design of the cofferdams for Gallions

Lock. Originally, both were intended to be block dams, but it was later decided that the outer dam should be a double sheet-pile wall cofferdam, largely because of the presence of the deep scour hole near the sill of the lock.

The outer cofferdam was designed on the basis of $\phi = 35°$ in the filling and an angle of friction of 18° between that material and the steel-piling, and with a factor of safety of 1·4 against failure under an extreme head of 42 ft of water above the old sill. But even with that by no means excessive factor of safety, the section (see Fig. 2b, Plate 1) appeared to be perhaps a little too generous. Nevertheless, after the lock had been emptied, the crest of the cofferdam moved inwards as much as 14 in., under a maximum head of 37 ft of water, and so it was felt that the design was not over-conservative. The movement was progressively built up, reaching the value of 14 in. in March 1954, after which no further increase took place. A head of 37 ft or thereabouts occurred only at high spring tides and by the time that head was first developed across the dam, in November 1953, the crest had moved 12 in.

Analysis of the inner block dam showed the desirability of placing a slab of tremie concrete between the dam and the sill of the lock. That was also thought, by some, to be erring a little too much on the side of safety, but when the block dam shifted in the manner described by Mr Dempsey everyone had been very relieved that there was that strut, or rather slab acting as a strut, which would stop the whole dam from sliding.

It might be worth emphasizing the point that the friction between the concrete blocks had been so much less than expected. Had it not been for the interlocks there seemed to be little doubt that at least the upper portion of the dam would have failed, with the possibility of somewhat catastrophic events in the Royal Albert Dock basin.

*** **Mr G. M. Cornfield** (Civil Engineer, The British Steel Piling Co., Ltd) observed that on p. 143 there was a statement that the outer line of piling of the sheet-pile dam was driven about 15 ft into the chalk. From the other details given it appeared that the inner line of piling of that dam had penetrated the chalk to a depth of 25 ft. What was the size of the hammer used for driving those sheet-piles, and could any other relevant details be given? Was the driving easy or difficult? Could the Author give any more information regarding the relative costs of the inner dam (constructed of block work), and the outer dam (steel sheet-piling and filling)?

Mr T. F. Burns (Consulting Engineer) asked why blockwork was used for the inner dam? It could hardly be considered less costly than a double-skin steel sheet-piled dam. He would have thought that the casting of 8,500 cu. yd of concrete in blocks, the careful placing and subsequent removal by diver, and the relative slowness of the work would have made the engineers seriously consider the employment of a sheet-piled structure. The possible use of horizontal girders transmitting the load to the end of the lock walls or the use of diaphragms would both appear to be less costly than the blockwork construction.

Could the Author give approximate costs of the two dams, the time of their construction and removal, and had the concrete blocks any salvage value?

During the discussion Dr Skempton revealed that both dams had moved under full hydrostatic head. Could the Author supply any further detailed information on the extent of movement in relation to tidal and impounded pressure? That data might be very useful to those who had to design such structures. Presumably the deep wells in the outer cofferdam functioned throughout the work.

The period of reconstruction of more than 4 years seemed considerable, particularly in view of the importance of the lock from a revenue earning aspect. One slide was shown illustrating night work—was that general or had it only applied to certain operations and on certain occasions?

The employment of ground-water lowering methods was interesting, in fact it seemed

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*** This and the following contribution were submitted in writing after the closure of the oral discussion.—Sec.

to be one of those jobs which would have been difficult to achieve successfully without the use of deep wells.

Apparently some settlement had occurred as a result of the deep-well pumping. Could the Author give some information as to the location and extent?

A rather cautious view seemed to have been taken by the Authority of the stability of the lock floor, since a margin of safety of 10-ft head from 82 to 72 was called for and, in addition, kentledge was placed on the floor. The kentledge apparently amounted to about 2·5 cwt/sq. ft of floor or the equivalent of about 4½-ft head of water. Did the Author consider that that latter provision was really necessary and if so would it not have been simpler to use ballast instead of supplying, placing, and removing kentledge, which he presumed was concrete blocks?

The Author had carefully set out the uses of the five grades of concrete but had given no corresponding mixes. Such information, with the crushing strengths specified, would be useful.

No particulars had been given as to the accuracy of the dressing of the quoin posts and meeting faces of the gates. There was often much difference of opinion as to the need for great accuracy in dressing of the greenheart; would the Author give his views on the subject and describe what was done in the present case?

Were any measures taken to overcome the effect of shrinkage of concrete skin on the walls and was the B.R.C. fabric placed near the exposed surface? To cast lengths of 140 ft in one pour seem to be excessive—had it proved entirely satisfactory?

Why were the Philbolts made as long as 3 ft? Presumably the function of those bolts was to secure effectively the facing to the body of the wall. Were any tests for adhesion made on the Philbolts and if so could the results be given?

The Author, in reply, said that a number of interesting points had been raised on matters which he had been unable to include in the Paper or had been prevented by restrictions of space from so doing. It would be realized that the Paper had been produced in September, while the work which it described had still been incomplete. However, the majority of the major construction work intended had, in fact, been done.

Mr Wild had asked whether, if the Author had to do the work again, he would do it in the same way. The Author felt that it was correct to say that the work which had been carried out at Gallions had proved very satisfactory and had shown that the original design, to which Mr Peel had referred, had proved to be perfectly sound. There had been a little concern until the dewatering had taken place, but after the preliminary excavations had been made the work had given no real cause for anxiety. To take away part of the floor of a lock, as had been done at the Gallions Entrance and to substitute another type of material, with uncertain shrinkage factors, had caused a good deal of anxiety until they had become accustomed to its behaviour; but he thought that, as all who had seen the finished work would agree, there was now almost a new lock, with a new wall face, new gates, and new machinery. That had been achieved at a cost which was only a proportion of what it would have cost to build a new lock in the same place under the existing conditions. Other things being equal, he would say, therefore, that if he had the job to do again he would make very little change from the way in which it had been carried out. A saving might be made on temporary works, but in general the major construction would be the same.

The deep-well system—had proved adequate—he would say even more than adequate, because although No. 7 well had never been used, and no pumps had been installed, the water plane in that area had been very little changed. If the work had to be repeated it should be possible to achieve a satisfactory result with a slightly wider spacing.

It was a little difficult to compare the conditions at Avonmouth with those at Gallions. The head of water was very much greater at Avonmouth than at Gallions and the velocities were very much higher. With regard to the discharge sluiceways of the outer apron, there were only two openings at Avonmouth as against six at Gallions. The Author had had, through the kindness of Mr Matheson, an opportunity to see the work carried out at

Bristol before undertaking the researches at Gallions. They had been fortunate in having that information to assist them. It was a little difficult to say that the methods of approach to the problem were quite the same, because at Gallions the amount of scour was not so serious as at Avonmouth, and they knew that on the removal of the outer dam they would be left with a very satisfactory rubble bottom as compared with the original chalk, which had proved to be soft. Also, owing to the lower water velocities, it had been thought, and he believed would be proved in practice later, that, if they could achieve a uniform flow over the apron by a minor modification of the sluiceways, they would have done all that was necessary to avoid future scour.

The matter really resolved itself into a question of economics. It would have been quite reasonable to assume, from the researches at Avonmouth, that all that was needed was to put in large amounts of rubble of good size at the scour-hole, or concrete blocks, or a pavement of some nature, and the water would not disturb it. If that had been done, however, it would have cost about £20,000. It would quite reasonably have prevented any further trouble, but what had in fact been done had been very much cheaper. The model had been built for about £1,700, including the cost of labour. The results found from the model were not, as Mr Allen had fairly said, to be considered a complete answer to researches on sluiceways, but they had given the information which was desired at Gallions, and putting that information into effect had cost approximately £3,000 to £4,000, giving a total cost of between £5,000 and £6,000 for all the work necessary to solve the problem. They had therefore saved about £15,000 compared with the cost of tipping in blocks, rubble, or large amounts of concrete, and that had been, from the point of view of an engineer, the most satisfactory way of dealing with the problem.

Mr Sivewright had asked for the rate of flow from the P.L.A. sluices. It could be taken as approximately 2,000 cusecs at maximum discharge. Measurements had been taken of the flow over the weir, illustrated in a slide shown by Mr Allen. The weir was situated at the far end of the outlet chamber. With regard to the altered time of locking by the construction of the sluices, they had not yet proved that point in practice, but he felt confident that by making a uniform flow through the sluices and sluiceways, and with the sluiceways all running full bore, the flow must be greater than would have occurred if sluiceways Nos 1 and 2 had been running full bore and the remainder only at partial capacity. He was of the opinion, therefore, that not only had they increased the output but the locking time was reduced by forming a truly even discharge and a satisfactory and uniform flow was achieved over the apron. By that means he thought that they had definitely added to the efficiency of the outer sluiceways.

With regard to the point raised by Mr Reed, they had had no results as yet on the operation of the machinery, because there had not been time to test it. The gates had been put into full use on Christmas Eve.

Mr Dempsey and Professor Skempton had referred to the blockwork dam. He agreed with Professor Skempton that they had been very pleased to have the horizontal interlocking sections in the blockwork. The friction between the blocks proved to be very low, as had been seen, and although he did not think that the Royal Albert Dock would have been the victim of a catastrophe, they had been glad to have something up their sleeve at that time. The original gates were still in position and would have proved effective. The water was being lowered and there was only about a 12 ft difference of head at the time that the dam started to move. There would have been time, he thought, to take all precautions to safeguard the Royal Albert Dock from being dewatered.

The Author thanked Mr Peel for his comments on the original work and also for having provided a very satisfactory design which the Author had had the pleasure and privilege to carry out.

The piling for the outer dam had consisted of Appleby Frodingham No. 3 section for the outer skin and No. 5 Larssen for the inner. A portion of the Larssen piles had been driven through the original scour-hole, and at the edge of that scour the penetration into chalk had reached 25 ft. Driving had not been exceptionally hard and had been carried out by a No. $9B_3$ McKiernan-Terry hammer slung from a 15-ton derrick. The rate of driving had averaged between 5 and 6 piles per day. The piles had been 70 ft long and

the labour involved in handling and pitching accounted for much of time required for driving.

With regard to the relative costs of the two dams, the inner dam could be taken as costing about £550 per foot run, based on the effective length of the dam being 198 ft. The outer dam of steel sheet-piling and filling had a mean length of about 250 ft and the average cost per foot run was about 20% less than for the blockwork dam.

One of the principal reasons for the adoption of the blockwork dam at the inner position was the reduced space required, which involved less encroachment into the shipping area. In addition, that construction had not been under tidal conditions, and the provision of steel girders to transmit the load to the wing walls, as suggested by Mr Burns, would have had to be carried out entirely by divers.

The concrete blocks which had been recovered from the inner dam were being utilized by the Port of London Authority for the construction of another dam of similar dimensions at the Millwall Dock.

The deep wells provided in the dams had not been utilized during the major part of the construction.

Night work had not been customary and the sodium lighting illustrated had been provided by the Contractor for normal winter conditions.

Some settlements had occurred as the result of deep-well pumping and the reduction of water level behind the lock walls. The settlements had been limited to the general ground level immediately behind the lock walls, and had reached 9 in. in places.

The placing of the kentledge on the floor of the lock had been decided upon as a safeguard and had had to be undertaken before the result of the deep well pumping could be proved. The lowering of the water level by those deep-well pumps had been more effective than had been anticipated and, in consequence, some of the kentledge had been additional to requirements.

Concrete blocks had been used in preference to ballast in view of the large amount of work necessary on the floor of the lock for the construction of the knees, which would have required the ballast to be closely confined.

Table 1 showed the concrete mixes and crushing strength specified.

TABLE 1

Grade	Minimum resistance to crushing: lb/sq.in.		Water/cement ratio	Volume of aggregate to 112-lb. cement, allowing for 25% bulking			Slump range
	7 days	28 days		Cu. ft All in	Cu. ft Coarse	Cu. ft Fine	
"A"	—	—	—	$12\frac{1}{4}$	—	—	1–2
"B"	900	1,500	0·70	$10\frac{3}{4}$	—	—	2–3
"C"	1,700	2,500	0·60	$8\frac{1}{4}$	—	—	2–4
"Cg"	1,700	2,500	0·60	—	5	$2\frac{3}{4}$	3–4
"D"	2,000	3,000	0·50	—	4	$2\frac{1}{4}$	4–7
"Dg"	2,000	3,000	0·50	—	$3\frac{3}{4}$	2	4–7
"E"	2,500	3,750	0·45	—	$3\frac{1}{2}$	2	4–7
"Eg"	2,500	3,750	0·45	—	$3\frac{1}{4}$	$1\frac{3}{4}$	4–7

The dressing of the concrete had been carried out to an accuracy of about 0·006 in. The granite work had been completely re-built and the conditions had been suitable for the best possible workmanship. Excellent conditions of gate tightness had been achieved and the Author considered the high degree of accuracy specified to have been justified.

No special measures had been taken to provide for expansion or shrinkage on the face of

the lock, for the majority of the work was under water, and the variations in temperature were limited.

The Philbolts had been made as long as 3 ft because of the poor condition of much of the concrete in the original walls. Tests had been carried out on three typical Philbolts which failed under test at 6·2, 4·2, and 8·2 tons respectively.

The closing date for Correspondence on the foregoing Paper has now passed and no contribution other than those already received at the Institution will be published.—SEC.

RAILWAY ENGINEERING DIVISION MEETING

10 January, 1956

Mr M. G. R. Smith, Member, Chairman of the Division, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Author.

Railway Paper No. 60

CONSOLIDATION OF BALLAST

by

* Idris George White, A.M.I.C.E.

SYNOPSIS

When expenditure on day-to-day maintenance of railway track is analysed, it is seen that the most costly item is packing the track. A further proportion of other track maintenance work can be directly attributed to unsatisfactory packing.

Before discussing how present-day methods of "hand" packing of track can be improved and speeded up, the functions and characteristics of ballasts are considered.

The history of the packing of track, known as tamping in the U.S.A., is traced through the past 100 years. Methods of packing of track at present in use on the London Midland Region are also described. In 1948, a mechanical tamper became available on British Railways, and its merits and limitations are discussed.

During 1954, smaller "off-track" tampers have become available, and in order to assess their possibilities comparative tests were made on the London Midland Region. These tests are described and a Table of comparable information is given, with a description of each of the tampers.

Reference is made to the possible use of tampers to assist the British Railways' modernization programme by releasing track labour for other work, and to curtail the overall time during which a speed restriction has to be imposed for re-laying and re-ballasting work. The use of tampers for maintaining track with long welded rails is discussed.

The Paper suggests that further research is now necessary on ballast to determine the optimum depths for both electric and diesel traction, and to decide on the size of ballast best suited to various mechanical tampers.

Introduction

Ever since man first began to follow well-defined paths on foot, he has been faced with the problem of preventing the natural surface from breaking up, so that he might continue to proceed with the minimum of effort. With the advent of the wheeled vehicle, heavier loads, and greater speeds, this problem became more acute. It was in an endeavour to overcome this very problem that railway track was evolved.

The steady introduction of heavier axle loads on railways, coupled with high speeds, has necessitated special attention to methods and materials that will maintain track with a high standard of longitudinal and cross levels. The Author proposes to review a principal factor to be considered in achieving this object; ballast and its consolidation. By ballast is meant the evenly graded, small-sized, artificial, or

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natural, material which is spread on the formation level to support the track and pack the sleepers.

Functions of ballast

It is suggested that ballast has the following functions:—

1. Filling any inequalities in the formation.
2. Supporting the track and distributing the pressure on the formation.
3. Resisting lateral displacement of the track.
4. Resisting longitudinal displacement of the track.
5. Resisting vertical movement of the track.
6. Draining the track.
7. Protecting, in some cases, the formation from frost.

1. *Filling any inequalities in the formation*

The formation on which a railway track is constructed is subject to variation both in shape and material.

The material varies from rock to clay, sand, or peat and consequently its shape varies from irregular to undulating, depending on the rate of settlement. There is also a variance in the depth of sleepers, chairs, and rails. All these inequalities can be compensated by varying the depth of ballast.

2. *Supporting the track and distributing the pressure on the formation*

A great deal of labour is required to make good the imperfections in railway track, the chief of which are due to the varying nature of the formation and the deformations produced by the track under load.

These imperfections could be lessened by concreting the track, at great cost; paradoxically, this would then have an adverse effect, leading to the rapid destruction of the track's component parts.

To prevent any considerable permanent deformation of the natural strata which have been trimmed to a fairly regular shape to carry the track, the increased pressure (due to loads passing over the track) must be distributed in such a way that the whole of the formation is uniformly loaded within its carrying capacity.

The unit carrying-capacity of the formation can be kept at the maximum by providing adequate drainage. The maximum unit pressure on the formation can be reduced by increasing the stiffness of the rail, by increasing the length, width, and number of sleepers, and by increasing the depth of ballast under the sleepers.

Theoretically, to obtain uniform loading of the formation, the ballast must uniformly distribute the load from each sleeper, and the rail, in turn, must distribute to each sleeper in the immediate vicinity an equal portion of the load.

This equal distribution of the load by the rail can only be achieved by the sleepers settling in the ballast sufficiently to allow the stiffness of the rail to transmit to adjoining sleepers their portion of the load, thus producing a flexure or wave motion in the rail.

Closer sleeper spacing produces a more uniform distribution of the load on the formation and, consequently, increases the total load which the formation will carry.

Too close a spacing of sleepers gives rise to practical difficulties in packing track. (Considerations of the factors to be taken into account in arriving at an optimum sleeper spacing and rail sections for particular conditions are outside the scope of this Paper.)

In a report to the International Railway Congress, 1909, Mr M. L. Myers suggested that an idea of the way in which ballast assists in spreading the load from sleeper to formation can be obtained by supposing the whole of the ballast to consist of 2-in. cubes. Considering a regular pyramid of these cubes, as shown in Fig. 1, he assumed that the load exerted by a cross-section of the sleeper 1-in. in depth was W and the pressure on each of the cubes of ballast was P, then $2W = 5P$.

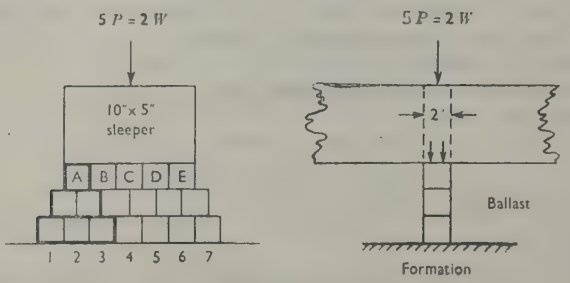


FIG. 1.—PRESSURE DISTRIBUTION

It was suggested that the pressure from cube A would be transmitted through the pyramid to cubes 1, 2, and 3 and thence to the formation in the proportions of $\frac{1}{4}$, $\frac{1}{2}$, and $\frac{1}{4}$; the pressure from cube B would be transmitted to the formation by cubes 2, 3, and 4. Similarly, the pressure on cubes C, D, and E would be transmitted in a like manner on to the formation through the respective cubes in the bottom layer.

To obtain the total pressure on the formation from the sleeper pressure on cubes A, B, C, D, and E, the individual pressures were combined.

Pressure from cube								
A	$\frac{1}{4}$	$\frac{1}{2}$	$\frac{1}{4}$					
B		$\frac{1}{4}$	$\frac{1}{2}$	$\frac{1}{4}$				
C			$\frac{1}{4}$	$\frac{1}{2}$	$\frac{1}{4}$			
D				$\frac{1}{4}$	$\frac{1}{2}$	$\frac{1}{4}$		
E					$\frac{1}{4}$	$\frac{1}{2}$	$\frac{1}{4}$	
Pressure from A + B + C + D + E =	$\frac{1}{4}$	+ $\frac{3}{4}$	+ 1	+ 1	+ 1	+ $\frac{3}{4}$	+ $\frac{1}{4}$	

Consequently, the total pressure from the considered section of the sleeper would be carried to the formation as follows:—

Cube No. 1 carries	$\frac{1}{20}$ =	5%	of $2W$ =	$2\frac{1}{2}$%	W/sq. in.
2	$\frac{3}{20}$	15%	,,	$7\frac{1}{2}$%	,,
3	$\frac{4}{20}$	20%	,,	10 %	,,
4	$\frac{4}{20}$	20%	,,	10 %	,,
5	$\frac{4}{20}$	20%	,,	10 %	,,
6	$\frac{3}{20}$	15%	,,	$7\frac{1}{2}$%	,,
7	$\frac{1}{20}$	5%	,,	$2\frac{1}{2}$%	,,

This distribution of pressure is over an area of 7 in. × 2 in. = 14 sq. in., and it is shown graphically in Fig. 2.

It is interesting to note in Figs 3 and 4, the pressure diagram produced by Myers for varying depths of ballast and for varying sizes of ballast. From these

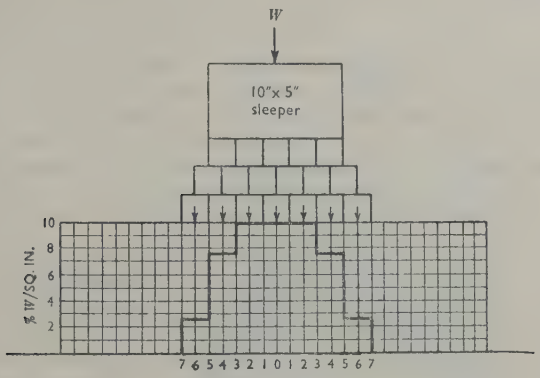


FIG. 2.—PRESSURE DIAGRAM

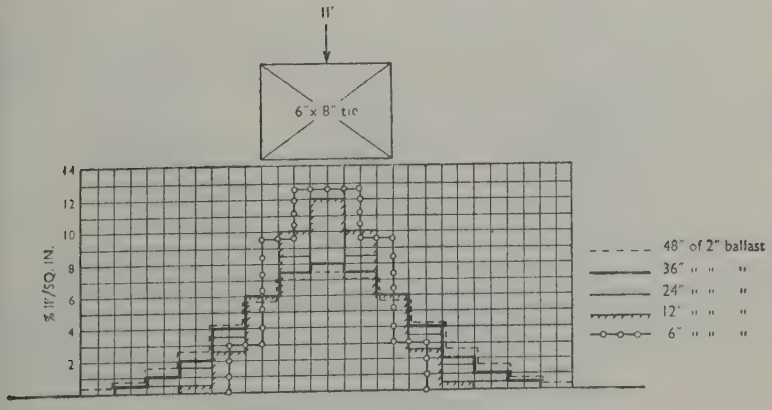


FIG. 3.—PRESSURE DIAGRAM FOR VARIOUS DEPTHS OF BALLAST

diagrams it can be seen that the unit pressure of the formation can be decreased by increasing both the depth and size of ballast.

The ratio in which the unit pressure is decreased by the increased depth of ballast is, however, a disappointing one, as shown in Fig. 3, a decrease of 40% in the maximum unit pressure being obtained by a 700% increase in depth of ballast.

The character of the pressure on a longitudinal section of formation can be seen by plotting a series of diagrams similar to Fig. 2 to the correct sleeper spacing.

The above can only be an approximation of actual pressures because so many indeterminable factors must be considered, but it is interesting to note how near these results are to those produced by more recent American research work,[1] as shown by Fig. 5.

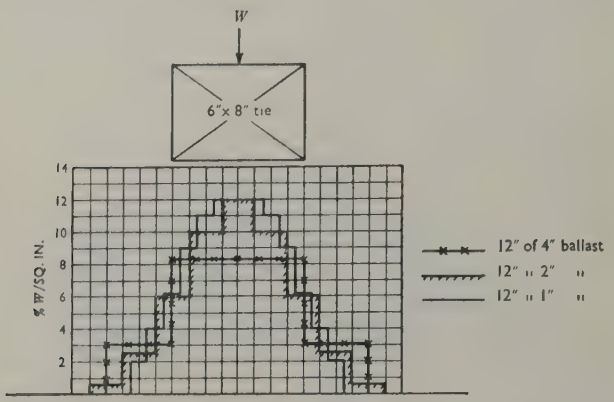


FIG. 4.—PRESSURE DIAGRAM FOR VARIOUS SIZES OF BALLAST

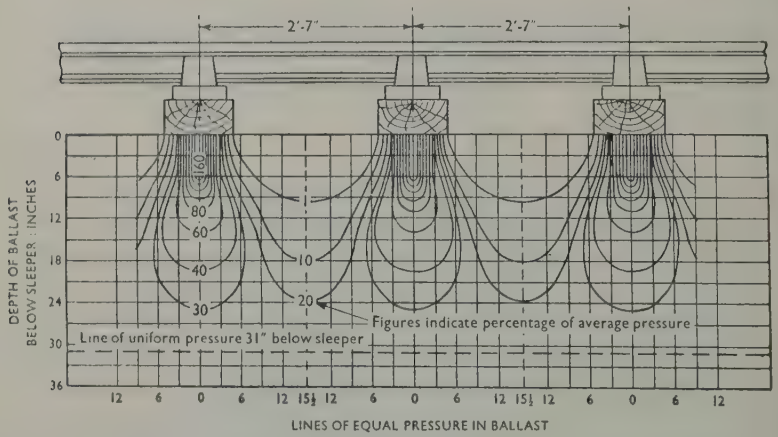


FIG. 5.—DISTRIBUTION OF PRESSURE FROM SLEEPERS

3. *Resisting lateral displacement of the track*

This function is a most important one, especially with the present development of long-welded rails.

It is a well-known fact that poorly ballasted track goes out of line much quicker than well-ballasted track where the frictional resistance of the ballast both under the

[1] The references are given on p. 188.

sleeper and, to a smaller degree, at the sleeper sides, coupled with the actual weight of ballast at the sleeper ends retains the track in line.

In the past, attempts were made to increase the ballast resistance at sleeper ends by laying boards along them and heaping ballast against the boards. This was discontinued for obvious maintenance reasons. However, it is evident that the better the packing of the sleeper, the greater frictional resistance there is to relative movement between the sleeper and the ballast.

Tests carried out by the Research Department of the British Transport Commission suggest that the track between bogies on long-wheel-base vehicles tends to lift as the vehicle travels. Consequently, the most important factor in averting lateral displacement of the track is the ballast shoulder. The tests indicate that a 7-in. shoulder is adequate, but the Author suggests that a 12-in. shoulder is more desirable on curves and where long-welded rails are installed.

Hence, ballast friction contributes to the concept of track as a Vierendeel girder.

4. *Resisting longitudinal displacement of the track*

Rails have a tendency to move in a longitudinal direction because of temperature variations, and also because of the frictional effect of adhesion between wheel and rail which occurs with braking or acceleration.

Ballast resists this tendency to move, partly because of friction between it and the sleeper, and partly because of the mass of ballast to be moved in between sleepers.

5. *Resisting vertical movement of the track*

Tests carried out on the former L.M.S. Railway have shown that if a timber sleeper rests on the ground and is not ballasted up, a force equal to the weight of the sleeper is required to raise it, but if the sleeper is well ballasted the force required to lift it is increased by approximately one-third.

This factor is an important one as sleepers do not become deformed to the same extent as rails when under load; consequently, provided that a flexible form of rail fastening is used, no excessive wear of the various track components will take place with well-ballasted track.

6. *Draining the track*

Sleepers and fastenings, if laid on an impervious bed, would soon show signs of rot and corrosion arising from dampness. Track circuiting would also be adversely affected. Hence, one function of ballast is that it must allow water to filter through.

Sleepers which lie in water soon begin to "pump," and slack joints and poor alignment follow.

7. *Protecting, in some cases, the formation from frost*

Where the formation consists of clay, it is desirable that it should be protected from frost as far as possible, especially if the drainage is not perfect.

It has been shown on the Continent that this can be achieved by minimum depth of ballast of 15 in.

Conclusions on the functions of ballast

It is evident that ballast, in order to fulfil the majority of its functions, must be well consolidated, but before methods of consolidation can be considered, it is necessary to decide on the type of ballast that can be used, together with any defects

it might possess. First, the ideal ballast should be hard, angular, irregular, with a good bearing capacity, and should pass through a 2-in. sieve.

The irregular, angular, material ensures the maximum frictional resistance, its bearing capacity ensures that it will not crush and the fact that it will pass through a 2-in. sieve ensures that there is no very large stone and, consequently, enables easy packing.

The further size specification for top ballast will be considered later.

Types of ballast used for railway track

Materials

The natural materials used for ballast may be divided into two groups; those which are found in compact masses, and those which are found already sub-divided into pieces of varying size.

The main types in the first group are granite, whinstone, and limestone.

Granite is regarded as the ideal ballasting material, because it has a high crushing strength and the individual pieces of stone are irregular and angular and do not have flat surfaces.

Limestone is extensively used for ballast, the best type being the blue stone. It has a lower crushing strength than granite. In the Manchester area, cases have been known of "down eyes" on viaducts being made up with an incrustation when limestone ballast has been used over a long period. This has necessitated washing them out with a diluted acid.

In the second group, river shingle, gravel, and sea shingle are used, but this type of ballast never seems to knit together as irregular-shaped ballast does; consequently, far more maintenance work is necessary.

Coarse sand is used as a ballast in some countries. Its greatest drawback is that it is liable to be blown away in hot, dry weather; it also tends to produce maintenance difficulties during long periods of wet weather.

Ash used for packing track is principally obtained from railway or colliery sources where it is a waste product of burning coal.

This type of ash is generally ungraded and contains a fair proportion of dust, large clinker, and other materials such as fire-bricks. Of recent years a much better type of graded coal ash has been obtained from electricity-producing power stations and it is a better packing material.

Ash ballast is used at places where colliery or brine subsidence occurs, and for packing sidings. It is always a useful material readily available for any washouts under the track.

Ash is an easy material to pack with, but it has the drawback of powdering easily and so causing a certain discomfort to track workers during hot weather. It also tends to cake during wet weather and thus retains water under the sleepers. It has a much lower frictional resistance than other materials to lateral movement, and where steel sleepers are used it causes corrosion under certain conditions. Track circuiting is more difficult on ash-ballasted track. B.T.C. Research Department tests have shown that ash ballast provides only half the resistance to lateral displacement of track that granite ballast does.

At one time furnace slag was looked upon as a waste product, but now it is regarded as a high quality product in itself, suitable for roadwork and railway ballast.

Some slags which have been specially treated have crushing strengths approaching that of the best granite.

It is essential that slag ballast should be free from slag dust, as the dust tends to form a hard coating on the formation, which causes uneven loading. Should the formation be a clay one, drainage difficulties are likely to follow. The majority of slags provide a higher resistance to lateral displacement of track than granite ballast.

In time all ballasts become impregnated with fine material from the atmosphere and from wagon spillage, and the rate at which this takes place depends on the geographical location of the track and the type of traffic passing over it. This has an adverse effect on track drainage and causes additional work in packing the track.

To overcome this difficulty ballast is either screened by hand, by off-track mechanical screens, or by an on-track machine such as the Matisa ballast cleaner. It is, of course, cheaper to replace ash ballast.

Almost every type of ballast, in time, allows the growth of vegetation which, if it is not removed, causes the ballast to disintegrate and sleepers to decay.

Economic considerations

Where several types of ballast are available for track work the questions of relative cost, distance of haul, and type of traffic using the track, must be considered.

On the London Midland Region, the Euston to Carlisle line has granite ballast, the former Midland trunk routes have limestone ballast, and the former Great Central has slag ballast.

Sources of each type of ballast are conveniently situated in relation to the lines.

Transport of ballast

Ballast is conveyed from the source of supply to the site of work in ballast trains made up of hopper wagons or wagons of the complete drop-side variety.

Hopper wagons are of the centre-chute or centre-and-side chute type, the latter being more desirable when formation blanketing work is being carried out.

The brake-van is sometimes fitted with a ballast plough which spreads the ballast more evenly.

We have so far considered ballast as being of a uniform size from the surface of the formation to the underside of sleeper, but in the past ballast was considered as two separate layers, known as top ballast and bottom ballast.

Bottom ballast was the larger stone which was laid on the formation surface, the stone being 6 or 9 in. Top ballast was the smaller packing stone which was then laid on the bottom ballast to facilitate easy packing of the track.

Bottom ballast was used extensively on lines constructed at the turn of the century, and Figs 6a and c illustrate typical cross-sections of track adopted by various railway companies in Great Britain at that time. It is significant that a great deal of formation and drainage difficulties have been experienced with many of these lines, especially those having a clay formation.

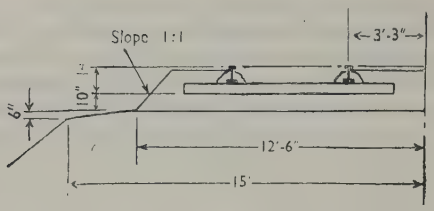
The present practice in the majority of countries is to use just the smaller-sized stone.

Methods of consolidation of ballast

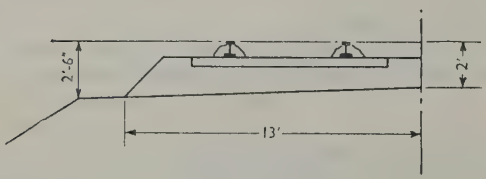
Historical

When track maintenance work is analysed it is seen that more than 60% consists of packing the track and a further percentage of other work results directly from defective packing.

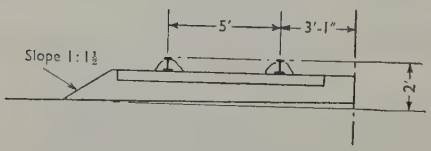
The Author suggests that 15 in. of ballast under the sleeper is both an ideal and an



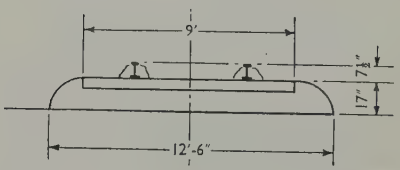
GREAT WESTERN RAILWAY
Crushed stone and slag, to pass 2-in. ring
Gravel



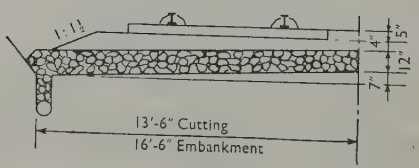
LONDON & NORTH WESTERN RAILWAY
Gravel, iron slag, cinders, broken granite, etc.
All sizes between ½ in. and 2 in.



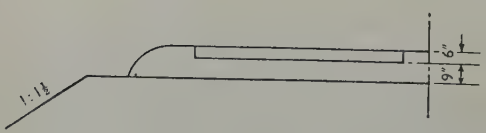
NORTH EASTERN RAILWAY
Cinders from coke-ovens and iron works



NORTH BRITISH RAILWAY
Slag, engine ashes, broken whinstone, and gravel

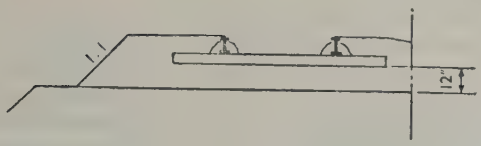


MIDLAND RAILWAY
Top ballast: granite clippings, broken slag, or gravel
Bottom ballast: granite or slag clippings, gravel, or ashes
Sizes from 1 in. to 2 in.

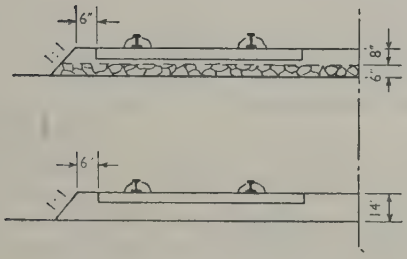


CALEDONIAN RAILWAY
Slag in pieces weighing not more than 6 oz.

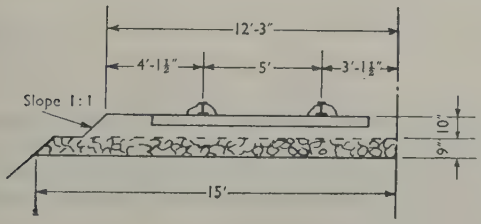
FIG. 6a.—TYPICAL CROSS-SECTIONS OF BALLASTED TRACK, *circa* 1900



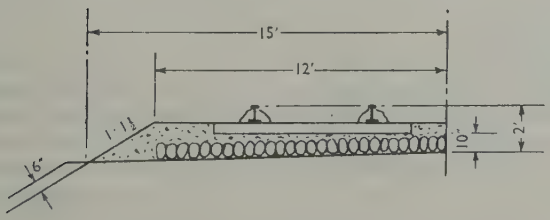
GREAT EASTERN RAILWAY
Gravel generally
Broken ironstone slag at water troughs



GREAT NORTHERN RAILWAY
(*above*) 1½-in. to 2-in. broken slag
(*below*) 6-in. broken slag
Screened 2-in. gravel



LONDON & SOUTH WESTERN RAILWAY
Gravelly ballast, screened
Also hard stone, broken to pass 2-in. ring



LANCASHIRE & YORKSHIRE RAILWAY
Clean cinders
Lower layer, broken stone, in water troughs, wet tunnels, etc.

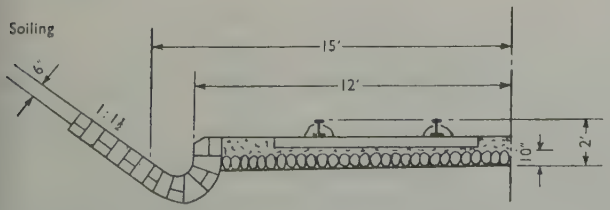
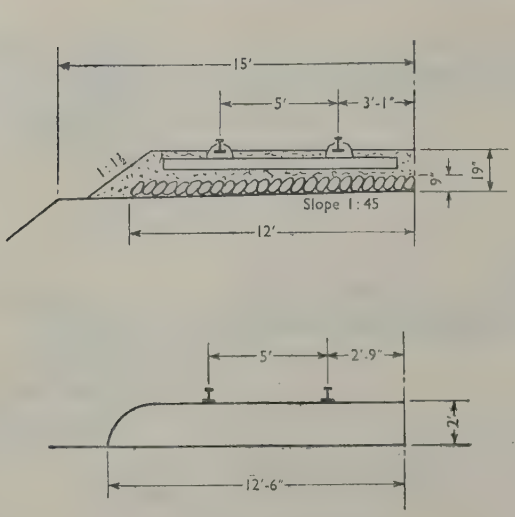


FIG. 6b.—TYPICAL CROSS-SECTIONS OF BALLASTED TRACK, *circa* 1900



GREAT CENTRAL RAILWAY
Top ballast: screened cinders, ashes, or gravel
Bottom ballast: hand-picked stone

GREAT NORTHERN RAILWAY (IRELAND)
Broken stone, gravel, or cinders

FIG. 6c.—TYPICAL CROSS-SECTIONS OF BALLASTED TRACK, *circa* 1900

economical depth, but unless it is well consolidated, it will be impossible to attain a good top and line to the track.

In 1860, one method of packing track was to lift the rails with wooden levers and ram broken stone under each sleeper with flat-ended hammers. This method must have been a most laborious one, producing poor results.

The next recorded method appears to have been for two men using shovels to work opposite each other each side of the sleeper, having first adjusted the track to the necessary height. Modifications of this time-honoured method are still in use today immediately following re-laying.

The use of an end-tamper was popular for some time in America and Germany. The end-tamper consisted of a 3-ft-8-in. spring-steel bar $\frac{3}{4}$ in. $\times$ $\frac{3}{16}$ in., having a $3\frac{1}{2}$-in. square handle, and a 3-in. flat, convex-end, specially-hardened tamping end. Level boards were set across the track which was lifted to the required height by jacks; a man would then lever a sleeper up by inserting a shovel under one corner of its end, pressing down with his right hand and at the same time using the end-tamper in his left hand to tamp along the length of the sleeper, as shown in Fig. 7, facing p. 186. It was said that this tool was popular as it obviated the necessity for opening out ballast from between sleepers, and was, consequently, two to four times faster in its work than any of the side-tamping tools and that, in the hands of an expert, better work was usually performed.

It is interesting to note that, about 1906, one of the first attempts was made to mechanize packing of track when end-tamping, by means of an air-blast, was tried, but without great success.

Side-packing or -tamping seems to have been the current practice elsewhere on the Continent at that time, using a "ramming bar" 5 ft 6 in. long, and $\frac{7}{8}$ in. in diameter with a sharp flat chisel at one end and with an inclined spade at the other. Here

again, the track was jacked up to the requisite level and two men would work packing from opposite sides of a sleeper, as shown in Fig. 8, facing p. 186.

The beater-pick in a variety of forms seems to have become popular at the turn of the century and is used to this day in some places, as shown in Fig. 9.

It is recorded in 1900 that "Sleepers are only packed for a distance of about 10 inches on both sides of the rails. Otherwise, if the packing under the rails were to work loose while the middle portion of the sleeper was still packed tightly, the sleepers would tend to rock when a train ran over the track; there would also be a risk of sleepers breaking. If the ballast is fine, it is recommended to pack more tightly towards the ends of the sleepers and less tightly towards the middle. If ballast consisting of slag is used, it must not only be packed under the rails but also, but not tightly, under the middle of the sleepers. It is better to pack the joint sleeper and the sleeper next to it more tightly."

It was also recommended that ballast should be renewed every 15 to 20 years.

Present-day methods of packing

In all cases track is packed working towards the direction of traffic. Before packing commences, care must be taken to ensure that all rails are bearing firmly on their chairs and base-plates and that all fastenings are tight.

Shovel-packing

In this method the predetermined level of the track is set by jacking up and two men pack from the opposite sides of a sleeper at a distance of a shovel-width each side, and under each rail, using the standard ballast.

This method is used immediately after re-laying, and, after going through the track two or three times, a reasonable top can be attained. The cross-level of the track is checked every three or four sleepers.

Fly-packing

After new ballast has been shovel-packed, slight slacks develop and these are adjusted by using chippings for packing. The chippings consist of stone graded from $\frac{1}{4}$ to $\frac{1}{2}$ in. By this method the ballast is removed from round the sleepers which require packing, and a quantity of chippings is spread under the sleeper after the track has been lifted by jacks. The quantity of chippings which is spread evenly under the sleeper, and only under the whole sleeper width for a distance of 15 in. each side of the centre of either or both rails as necessary, is gauged by experience; hence, this method can produce an indifferent longitudinal level when carried out by inexperienced men.

Measured shovel-packing

Measured shovel-packing was introduced more recently, and is still used extensively in Great Britain and on the Continent, especially in France, away from long-welded rails. The method presupposes that the bulk of the bottom ballast has been consolidated and that minor depressions in longitudinal rail-level and cross-level can be accurately measured and the deficiencies exactly made up by introducing a measured quantity of fine chippings spread evenly under the sleeper for a distance of 15 in. each side of the rails after the track has been jacked up.

In addition, sleeper- or rail-depressions in the case of the stiffer F.B. rail are

measured under load by void-meters calibrated to the same scale as the sighting-boards used for longitudinal measurement.

For good results this method is dependent on three factors: that there is a good depth of clean ballast under the sleeper, that any depressions greater than ½ in. shall have been made good, and that all fastenings shall have been tightened up before any measurements are obtained.

It is unfortunate that this method makes such a great demand on manpower and that it is sometimes difficult to get platelayers to fully understand the methods of measuring the required quantities of chippings. All too often the ballast from between sleepers is left opened out for days on end, thus facilitating longitudinal and lateral displacement of the track.

In view of the present policy of joint elimination, this could have serious consequences. The smooth surface produced by the measured shovel-packing chippings has been regarded as giving sleepers less frictional resistance to longitudinal and lateral stresses produced in rails, but it is suggested that, provided the bottom ballast is clean and well consolidated and the track fully ballasted up with adequate shoulders, there is little fear of distortion.

With the commencement of the British Railways' modernization programme substantial demands can be expected on an already depleted number of "skilled" platelayers for look-out duties. When the Manchester–Sheffield–Wath electrification was carried out, this amounted to 30% of the available manpower on that line.

Consequently, as such a high proportion of time is spent on packing track, manpower savings can be made by resorting to mechanical tamping.

The modernization programme will also necessitate additional track occupations, and if trains are to keep their present timings it will be necessary to resort to means of removing speed restrictions necessary for re-laying and re-ballasting work earlier than at present.

The present policy of joint elimination also requires the adoption of a technique for packing track without opening out the ballast, thus avoiding any possibility of distortion. This can only be carried out mechanically.

There are therefore three clear objectives to be achieved by introducing mechanical tampers:

1. The saving of manpower.
2. The earlier removal of speed restrictions necessary for re-laying and re-ballasting.
3. The maintenance packing of track having long welded rails.

The first objective can be achieved by introducing any type of mechanical tamper known today; all types effect a saving in manpower and produce a uniform degree of consolidation of ballast.

The Author is of the opinion that the second objective can be achieved by first tamping newly laid ballast with a vibrating tamper, and then using a percussion tamper to consolidate the top 4 in.

The third objective can be achieved by using a percussion-type tamper.

Types of mechanical tampers available

A description of the types of mechanical tampers is given in Appendix I.

They can be divided into two main groups: "on-track" and "off-track" tampers. It is unfortunate that, in the industrial regions of British Railways which are mostly

concerned with modernization works, the use of an "on-track" tamper on trunk routes is limited. "Off-track" tampers will therefore have to be used to a considerable extent.

Several mechanical aids to tamping became available in 1954, and it was decided that a test should be made to assess them.

After careful consideration it was decided that a suitable place to make the test was on the Euston-Carlisle line immediately south of Tamworth. Here there are four tracks, two fast and two slow, running from a slight cutting on to an embankment.

The ballast at this place was granite, varying from 6 to 12 in. in depth, the track was 113 and 109 lb. F.B. rail with elastic-spike fastenings and Douglas-fir sleepers. There are seventy-five booked trains over this line per 24 hours, including thirty-five express trains. The speed limit on this portion of the line is 90 m.p.h., but the average recorded speed is about 75 m.p.h.

A control length was chosen for comparison purposes and fast hallade runs were made over the line before the tests were made. Also void-meter readings were taken throughout the portion of track concerned.

The test was made using the following mechanical "off-track" tamping appliances:—

Percussion	*Vibrating*
Kango	Wacker
Bosch	Jackson
Air compressor using { Broom-Wade tampers; Holman tampers; Consolidated Pneumatic tampers }	Vibromax

One set of percussion and one set of vibrating equipment was used each week by two teams of four platelayers, who had been trained by the makers for one day in the handling of the equipment.

The second day was spent in tamping a length of the down fast line, records of time and fuel consumption being kept, and the ease of handling the equipment and the men's reactions were noted.

The third day was spent in tamping switches and crossings, and each type of tamper was used to tamp a different sleeper which was afterwards removed so that the pattern of the tamping could be seen.

A summary of the results is shown in Table 1.

The test was made in the worst possible weather conditions, and the results certainly cannot be regarded as conclusive, but give a general indication of the capabilities of the appliances.

It would seem that line-side tamping with compressed-air hammers has definite limitations on British Railways with the present design of the generating unit, which is too large and heavy in proportion to its power output. The hammers also tend to freeze up quickly in cold weather.

The Kango tamper in its present form is too slow for plain line work, but is satisfactory for tamping switches and crossings, this being the purpose for which it was designed. The test did, however, show that all tampers consolidated the track to a uniform extent, but it is obvious that further experiments are necessary.

Fast hallade runs were taken at regular intervals after tamping, but the total distance tamped was too short for any useful information to be obtained beyond the fact that there was a general improvement.

TABLE 1.—COMPARATIVE TEST OF TAMPERS

	Vibromax	Wacker	Jackson	Bosch	Kango	Air compressor	Weather
Type of tamper	Vibratory	Vibratory	Vibratory	Percussion	Percussion	Percussion	Very cold with sleet, heavy rain, and showers
Weight of tamper	73 lb.	68 lb.		87 lb.	27½ lb.	34 lb.	
Weight of prime mover	352 lb.	314 lb.		600 lb.	400 lb.	560 lb.	
(a) Type of fuel	Petrol	Petrol	Petrol	Diesel	Petrol	Diesel	
(b) Fuel consumption	2·6 pints/hour	2·5 pints/hour	3·5 pints/hour	2 pints/hour	2·5 pints/hour	2 pints/hour	
Degree of noise	Quiet	Fair	Noisy generator	Noisy	Noisy	Very noisy	
Manœuvrability	Good	Good	Good	Good	Good	Fair, except in very congested areas	
Instruction time	8 hours	8 hours	8 hours	8 hours	8 hours	8 hours	
Men's reaction	Favourable	Favourable	Unfavourable	Favourable	Favourable	Unfavourable	
Type of track	109 lb. F.B.	109 lb. F.B.	109 lb. F.B.	113 lb. F.B.	113 lb. F.B.	113 lb. F.B.	
Type of ballast	Granite	Granite	Granite	Granite	Granite	Granite	
Speed of tamping P.L.*	69 sleepers/hour	62 sleepers/hour	60 sleepers/hour	48 sleepers/hour	29 sleepers/hour	18 sleepers/hour	
Speed of tamping P. & C.*	← Speed of tamping P. & C. approx. ⅓ that of tamping P.L. →						
Adaptability for tamping close sleepering	Good	Good	Good	Good	Good	Good	
Ease of moving loads over P. & C.	Good	Good	Good	Good	Good	Heavy	

* P.L.—plain line. P. & C.—points and crossings.

TABLE 2.—FIRST ASSESSMENT OF TAMPING

Make	6-ft rail			Cess rail		
	% Improved	% No change	% Worse	% Improved	% No change	% Worse
Vibromax	60·0	25·0	15·0	40·5	57	2·5
Bosch	63·5	31·5	5·0	47	53	0
Wacker	41·0	21·0	38·0	37·5	60	2·5
Kango	33·0	26·5	40·5	36·5	61·5	2
Jackson	44·0	25·5	30·5	20·5	73	6·5
Holman	46·5	37·0	16·5	15	72	13
Consolidated Pneumatic	48·0	32·0	20·0	35·5	50	14·5
Broom-Wade	49·5	35·5	15·0	54	46	0

TABLE 3.—SECOND ASSESSMENT OF TAMPING

Make	6-ft rail			Cess rail		
	% Improved	% No change	% Worse	% Improved	% No change	% Worse
Vibromax	57	14	29	38	50	12
Bosch	61	20	19	44	52	4
Wacker	39	20	41	35	56	9
Kango	30	25	45	31	57	12
Jackson	32	25	43	20	70	10
Holman	40	36	24	14	73	13
Consolidated Pneumatic . .	43	32	25	33	48	19
Broom-Wade	45	36	19	48	44	8

FIG. 7.—TAMPING WITH END-TAMPING BAR

FIG. 8.— TAMPING WITH SIDE-TAMPING BAR

FIG. 9.—TAMPING WITH PICK

FIG. 10.—VIBROMAX TAMPER

Fig. 11.—The Wacker tamper

Fig. 12.—The Bosch tamper

FIG. 13.—KANGO HAMMERS

FIG. 14.—L.M.R. LINE-SIDE COMPRESSOR

Void-meter readings were also taken at regular intervals and it is interesting to note the comparison between those taken immediately after tamping and those taken 4 months later, as shown in Tables 2 and 3.

Although the highest initial consolidation of ballast was achieved using the Broom-Wade pneumatic tampers operated from the L.M.R. line-side compressor, the best all-round results were achieved by the Bosch tampers. Good results were obtained with the Vibromax tampers.

The Bosch tampers also gave the best results for tamping switches and crossings.

The Jackson machine on test was about 10 years old, and is said to be a Canadian machine and not the same type as those at present produced in the U.S.A.

Consolidation of formation

No consideration has been given to the compaction of the formation on which the ballast is spread, as it has been assumed that this has been achieved in a uniform manner by the weight of traffic which has passed over the track since the line was constructed.

There are, however, cases where the formation has had to be strengthened by re-grading, draining, and blanketing with fine granular materials. In such cases it is essential that a high degree of compaction of the blanketing material is achieved before ballasting is commenced, and it is of interest to note that some of the best results yet recorded[2] were obtained using a Vibromax compactor AT 5000.

This apparatus, weighing 1,500 kg, consists of a heavy base on which an eccentric weight system is mounted (vibrator unit). When these weights are driven by a 10-h.p. diesel engine which is flexibly mounted on the base, a rectilinear oscillating force of 5,000 kg is produced. Depending on the position of the vibrator-unit, the force can, within limits, be deviated from the vertical direction to give a forward or a backward motion.

Economic considerations

A recent German Federal Railways' Report has stated that one Matisa tamper gives an average annual output equivalent to the labour of thirty-five men. After taking into account all charges in connexion with the machine, the following figures are quoted:—

Cost of hand-tamping	£664 per mile
Cost of Matisa tamping	£230 „
Saving	£434 „

The average output of the machine was 65½ miles, and the resultant annual saving £28,474.

Further research

The Author feels that much further research work is necessary on several aspects of ballast consolidation and suggests that, whereas experiments have been undertaken to find the optimum depth of ballast, no conclusive results have yet been obtained.

It would be of great value if tests could be made to determine the optimum depth of ballast for diesel and electric traction. Tests could also be made with various sizes of ballast for packing track laid with concrete sleepers to determine the best combination of size of ballast for each type of mechanical tamper or combination of tampers.

Consideration could also be given to testing vibrating tampers for compacting the ballast between sleepers, thus giving greater support to the ballast consolidated under the sleepers.

Acknowledgement

The Author wishes to record his thanks to Mr J. Taylor Thompson, M.C., M.I.C.E., Chief Civil Engineer, London Midland Region, for his permission to present the Paper.

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APPENDIX I

THE VIBROMAX MECHANICAL TAMPER

This is an "off-track" tamper of the vibratory type (Fig. 10) which can be powered either from its own mobile generator or directly off a mains supply. In the latter case special earthing arrangements are made. The portable generator can work four tampers at 110 V. Each tamper weighs 73 lb. and vibrates at a rate of 2,800 beats per minute. By an easy adjustment the tamping force can be varied from 0 to 1,000 lb. to accommodate varying kinds and sizes of ballast. No sense of vibration is conveyed through the handle to the operator.

The tampers are paired two rights and two lefts; the tamping blade is inserted vertically at the side of the sleeper a few inches from the rail and by slightly tilting the side of the handle farthest from the sleeper the blade works its way under the sleeper.

The operator can, after a little practice, feel when consolidation has taken place. Any ballast deficiency is made good by "pushing" more ballast under the sleeper. It should be noted that two men can work diagonally opposite each other either side of the rail.

Approximately 65 ft of cable is supplied with each hammer and it is possible to tamp up to seventy sleepers per hour with one set of four tampers.

The generator can be supplied with either a diesel or petrol engine, the former being preferable where railways are concerned in spite of the increased weight of the prime mover, as diesel fuel can be conveyed by normal rail services. A useful feature on this generator is that all four tampers can be shut on or off simultaneously from the power unit when warning is given of an approaching train.

Separate vibrating blades are available for tamping switches and crossings.

THE WACKER VIBRO-TIE-TAMPER

This too is a vibrating "off-track" tamper (Fig. 11) which can be worked from a mains supply or from a mobile generator.

The generator, which is powered by a petrol engine or a diesel engine, is capable of operating four tampers simultaneously.

The tampers are activated at an oscillation frequency of 3,000 per minute. The tamping action is similar to that of the Vibromax, two of the tampers being right-handed and two left-handed.

An unusual feature with this tamper is that tamping can be carried out up to 230 ft from the generator, the power being conveyed by 164 ft of cable wound off a drum to a power distributor and thence by 65-ft cable-lengths to each tamper.

The method of operation is similar to that of the Vibromax tamper. Interchangeable vibrating blades are available for switches and crossings.

THE JACKSON TIE-TAMPER

This is a vibrating "off-track" tamper like the Vibromax and Wacker, and it is operated and powered in a similar manner. Since it is a tamper originating in the U.S.A., it has not been available for test in Great Britain, and no details can be given.

THE BOSCH TAMPER

This is a percussion "off-track" tamper (Fig. 12) which can be operated from a mains supply with special earthing arrangements, or from its own mobile diesel generator, which can power four hammers simultaneously.

The Bosch tie-tampers can be supplied with a universal motor or with a high-frequency motor at 200 c/s. The advantage of the high-frequency motor is that it is much smaller than a universal motor producing the same output, and maintenance costs are very much less as there is no commutator and no brushes.

A unique feature is that, while the tamping hammer is being used, the weight of the tamper is taken by a telescopic stand.

The hammer heads can be supplied in various sizes and are specially shaped to take varying sizes of ballast and fit switches and crossings.

The weight of the blow is slightly less than that of a hand-beater, which ensures that the ballast will not break up.

The generator is powerful enough to supply current for site lighting, and at the same time power four hammers, in the event of night work.

The tampers produce a high degree of consolidation for both plain line and switches and crossings.

THE KANGO ELECTRIC BALLAST TAMPER

This is a percussion "off-track" tamper (Fig. 13) which tamps at the rate of 1,300 blows per minute. The tampers can be powered either by mains supply or from mobile 1-kW or 3-kW generators which will power two or four tampers respectively. The generators can be either petrol- or diesel-driven.

The tampers are comparatively light, weighing 27½ lb., and different-sized heads can be supplied for tamping stone or ash. These tampers are capable of lifting track.

THE MATISA TAMPER

This is an "on-track" tamper which can travel along the track under its own power and carries derailing equipment in a trailer. The tamper can be used "in between" trains, but is more often used under a total occupation of the track to be tamped.

Sixteen spades are operated in pairs, two pairs on each side of each rail; the depth of the rail is taken into consideration before tamping commences.

The spades are lowered and forced through the ballast by compressed air, and then each pair is drawn together and vibrated at the same time.

A high uniform standard of tamping can be achieved by a skilled operator.

A two-depth tamper has just been produced by the manufacturer and extensive trials are now taking place.

THE LINE-SIDE COMPRESSOR

The L.M.R. have developed a line-side compressor (Fig. 14) producing 30 cu. ft of free air per minute which can power two pneumatic percussion tampers. The compressor unit has twin storage cylinders and is mounted on rubber tires and can be taken along the cess to the site of work.

More recently a prototype "split-up" compressor has been developed; this has a separate engine and compressor unit which can be easily assembled. Separate twin-mounted air receivers are also mounted on rubber tires to assist in mobility, and the heaviest individual unit weighs 3 cwt.

This compressor is also capable of working two tampers.

APPENDIX II

SPECIFICATION FOR BALLAST.—L.M.R.

Granite ballast

The ballast shall be of the same quality as that used for the best road metal, of a quality approved by the Commission's Civil Engineer or his representative and in accordance with the sample submitted.

It shall be cubical in shape, not flaky, and free from dust.

It shall consist of a mixture of the following sizes expressed as percentages by weight:—

The whole of the ballast shall be capable of being passed through a ring 2 in. in internal diameter, of which not more than 1% shall be capable of being passed through a sieve with $\frac{1}{8}$-in. mesh.

Not more than 10% shall be capable of being passed through a sieve with $\frac{1}{2}$-in. mesh.

Not more than 40% shall be capable of being passed through a ring 1 in. in internal diameter.

90% shall be capable of being passed through a ring $1\frac{1}{2}$ in. in internal diameter.

Limestone ballast

The ballast shall be good hard limestone (not top rock limestone) of a quality approved by the Commission's Civil Engineer or his representative, and in accordance with the samples submitted.

It shall be clean and free from dust.

It shall consist of a mixture of the following sizes expressed as percentages by weight:—

The whole of the ballast shall be capable of being passed through a ring 2 in. in internal diameter, of which not more than 1% shall be capable of being passed through a sieve with $\frac{1}{8}$-in. mesh.

Not more than 10% shall be capable of being passed through a sieve with $\frac{1}{2}$-in. mesh.

Not more than 40% shall be capable of being passed through a ring 1 in. in internal diameter.

90% shall be capable of being passed through a ring $1\frac{1}{2}$ in. in internal diameter.

Slag ballast

The ballast shall be made from good hard slag of a quality approved by the Commission's Civil Engineer or his representative and in accordance with the sample submitted.

It shall be clean and free from dust.

It shall consist of a mixture of the following sizes expressed as percentages by weight:—

The whole of the ballast shall be capable of being passed through a ring 2 in. in internal diameter, of which not more than 1% shall be capable of being passed through a sieve with $\frac{1}{8}$-in. mesh.

Not more than 10% shall be capable of being passed through a sieve with $\frac{1}{2}$-in. mesh.

Not more than 30% shall be capable of being passed through a ring 1 in. in internal diameter.

90% shall be capable of being passed through a ring $1\frac{1}{2}$ in. in internal diameter.

The Paper, which was received on 11 October, 1955, is accompanied by eight photographs and six sheets of diagrams, from which the half-tone page plates and the Figures in the text have been prepared, and by two Appendices.

Discussion

The Author introduced the Paper with the aid of a film.

Mr J. Taylor Thompson (Chief Civil Engineer, London Midland Region, British Railways) observed that the title of the Paper "Consolidation of Ballast" at once gave rise to the question of why that should not be taken to the logical extreme of consolidating and in fact solidifying the ballast, and having in effect a tram track. As mentioned in the Paper, there had been an experiment on those lines near Detroit in America about 30 years ago. A short length ($\frac{1}{4}$ mile) of solid concrete track had been laid, the argument being that the saving in track attention would pay 10% on the capital cost of laying the solid track. It had been 21 in. thick and, he thought, 10 ft wide, laid in slabs, and had been, of course, a rigid track designed to take the loads. It had a girder, a reinforced beam, under each rail. After about $2\frac{1}{2}$ years the engineers had been highly satisfied with it and thought that it proved their point that a solid track was an economic proposition. It cost the equivalent of £9,500 per mile, which was rather less than the present-day cost of plain track in the United Kingdom and showed the change in prices which had taken place since that time. In the period of $2\frac{1}{2}$ years, however, it had gone down an average of $1\frac{1}{4}$ in. and a maximum of 2 in., but the American engineers were quite satisfied—which seemed rather odd.

His recollection was that a few years later there had been an account of a derailment on that track, and it had given rise to so many difficulties that the American engineers had decided to remove it altogether. So far as he knew that experiment had never been repeated, and it looked as though ballasting as it was now known was likely to hold the field. A few years after the American experiment, however, what appeared to be a natural sequence had been tried in Britain. About 1930 someone had had the idea of trying not quite a solid track but the effect of putting in tar macadam as a foundation. It had been laid with a camber and former pieces put in to shape a bed for the sleeper, which was set partly into the tar macadam, the camber giving it a certain grip. A wedge-shaped piece of timber had first been rolled in; after removal of that timber the sleeper had been put in its place and the track laid. It had looked very nice on the first day, and the water ran off it very quickly. Photographs taken just after it was laid had shown that it looked quite presentable, but a little later it had had neither top nor line, and in a very short time that experiment had also disappeared. There was evidence, therefore, that solid track was not successful, and the half-solid, tar macadam idea was equally useless. He knew of no other tests going on in the world, and it appeared that the conclusion had been reached that stone ballast of the usual type must be taken as the standard form of track foundation.

The Author had dealt with the functions of ballast, one of the most important, of course, being distribution of loading. Fig. 5 showed that when the depth of ballast below the underside of the sleeper was equal to the spacing of the sleepers centre-to-centre, the pressure on the formation was about uniform. It was very nice to have uniform pressure on the formation, but he did not imagine that a stage would ever be reached when ballast would be deliberately made 2 ft 7 in. deep below the underside of the sleeper. As the Author had suggested, perhaps half that depth would be more likely.

There was the point, of course, that the strength of the formation in compression was some measure of the depth of ballast required. The weaker the formation the deeper the ballast, so that the pressure would be more uniformly spread over it.

Ordinary well-packed track would remain for 2 or 3 years in perfectly good order with measured shovel-packing, but it was interesting to recall that, at the time when he had first become interested in track, beater-packing had been the recognized method of packing (except, of course, on ash ballast, which had almost been forgotten now but which had formed a large part of the ballasting for trunk lines in Great Britain before and at the end of the first world war, and certainly for a large part of the East Coast main line; apart from ash, stone had been beater-packed). There seemed now to be a return to the same fundamental idea.

Measured shovel-packing was more scientific than beater-packing, which simply used brute force to break up the bed and force it against the underside of the sleepers. Advocates of measured shovel-packing disagreed and contended that, having a hard bed sufficient hard chippings should be used to raise the track by the right amount. The idea had originated in Britain and the French had developed it by producing true measured shovel-packing. Britain had then adopted the French development and scientifically it was probably the best thing to do; but developments in mechanical packing seemed to be leading back to beater-packing—but by mechanical means. The ultimate outcome was likely to be a form of mechanical packing probably, as the Author suggested, by means of a vibratory machine for the lower ballast, and a percussion machine (in effect beater packing) for the upper ballast.

Mr G. D. S. Alley (Assistant Engineer (Permanent Way), Western Region, British Railways) said that, in addition to the different kinds of material, two separate functions were involved. First, there was the requirement to consolidate ballast after relaying, and secondly, there was the ordinary maintenance consolidation, when it was regularly necessary to give a ½-in. or ¾-in. lift to the track.

Considering the first of these, in Germany the usual practice after obtaining occupation was the mechanical cleaning of the ballast followed by the removal of the old track and consolidation of the bottom ballast, first of all with vibratory tools and heavy rollers, making certain at the same time that the cross-level was adequate and that there was sufficient ballast. The resulting surface was so good that it required only a 2-in. layer to spread by mechanical means, giving a really firm bed which required no further consolidation for several years. The sleepers were simply laid on that bed and the top ballast applied over or between the sleepers. The French, on the other hand, considered that that gave much too solid a track and they preferred to clean the ballast in the normal way with a heavy cleaner and then allow the traffic to consolidate it. Such long occupations would be difficult in Britain, but it seemed to Mr Alley that with the current general increase in Sunday traffic some method of that nature might have to be considered.

Mr Alley thought that the L.M.R. stone specification given in Appendix II allowed too high a proportion of small stone; it was provided that not more than 40% should be capable of being passed through a ring of 1 in. internal diameter. The Western Region specified that the stone must be between 1¾ in. and 1 in. That size had to be taken into account when deciding in what way the ballast was to be consolidated. In France and Germany the stone was, generally speaking, considerably larger.

Before the London Midland Region tests referred to by the Author it had been agreed that certain tests would be carried out on the Western Region, in October 1954. Unfortunately, the Bosch agents had been unable to obtain any of their equipment in Britain in time for those tests, which had therefore been confined to maintenance tests on packing, Vibromax, Kango, and the on-track Matisa tamper.

A site was chosen where the bottom ballast was well consolidated but the formation was not good and the track required topping. That was a few miles south of Taunton on the up main line, where speeds were normally fairly high; a 36-chain length had been marked out, allowing 9 chains for each test. Hallade records had been taken both before and after the test, and void-meter readings prior to and immediately after the test, and 7 weeks and 6 months after the test. Three significant results had been obtained.

First, the tests yielded a comparison of the man-hours required for the different methods of working. The Matisa tamper was an on-line tool, but occupations had been available because work had been in hand in the vicinity. In those circumstances it took only half the number of man-hours required for measured packing. The men were unused to the Vibromax tools but took only slightly longer than with measured packing. They had been familiar with the Kango hammer for packing fitting work, but on reaching plain line their first effort had been so poor that they had to go over the 9 chains again and the packing had taken considerably longer than with the other methods. It was not suggested, of course, that the Kango was ideal for plain-line work, but it had been considered useful to test it for comparison with the other tools.

The second result concerned the effectiveness of the methods. Matisa, Kango, and Vibromax had given comparable results at the end of 6 months. The percentage of voids at that time was about 50% of those which had existed prior to the track being treated.

The third result was an economic one. An on-track tamper such as the Matisa was a costly tool, and it was necessary to make a heavy allowance of about £2,000 per annum to wipe off the capital outlay on it. Further, there was the question of the time available in the section. If it was necessary to have a gang standing idle while plant had to clear a section by running through or even by utilizing a site turntable, the cost of the idle time would soon add up to a considerable sum. It had been found from calculations made after the tests that if the tamper was not used for more than about 60 miles per annum, owing to the fact that it was waiting for traffic or occupation, it could actually use more man-hours and cost more than the Vibromax for the results obtained.

With regard to lateral stability, the Author had referred (p. 175) to the advisability of having a 12-in. shoulder. Mr Alley agreed that that was very desirable, but the Author seemed to suggest that the most important factor in averting lateral displacement was the ballast shoulder, but other tests carried out by the Research Department seemed to refute that. In Mr Alley's view, which he thought was borne out by the R. Dept tests, quite a large proportion of the resistance to lateral movement was provided by the ballast in the cribs.

Also on p. 175 the Author had referred to the temperature ranges on the Continent requiring a depth of 15 in. of ballast to obviate the formation being attacked by frost. Rather than go to a depth of 15 in. Mr Alley would like to see a 12-in. shoulder as the minimum, so as to spread the protection over the formation. Since the temperature range in Britain was much less than on the Continent he thought that British Railways would get more benefit from the use of ballast in that way; further research was needed on that point and also on the question of formation.

Mr A. H. Toms (Research Assistant to the Chief Civil Engineer, Southern Region, British Railways) expressed interest in what Mr Taylor Thompson had said about concrete track, because, he said, it seemed to him that an ideal formation should be elastic but non-yielding plastically. If it were possible to get all the resilience required from the track components, theoretically at any rate it should be possible to construct such a track and make it work. The breakdown which had occurred had probably been due to imperfect elasticity of road-bed support gradually leading to breakdown.

So far as the uniform loading of the formation was concerned, many diagrams of the same character as those given in the Paper had been produced, and the Americans had taken the matter up a good many years ago; but anyone who had made calculations in relation to soil mechanics formation problems would realize that uniform load-application to the formation was a theoretical and practical impossibility, and it was necessary to have a compromise. The logical approach seemed to him to be to find out what the soil would bear and then to design a road-bed which, elastically, would transmit the pressures and shear stresses, both longitudinally and transversely, which that soil would support.

At present there were still some uncertainties in estimating what the soil would support, and a primary requirement was to design further apparatus to carry out tests on soil under fluctuating shear stress. This was in hand. A member of his staff had made calculations which showed the maximum fluctuating shear stress under the track occurred at a depth of the order of 50 in. That was not the depth of maximum shear stress but it was the depth at which the range of shear stress was the greatest. There was no certainty that the method which he had developed for determining track blanketing depth on the basis of maximum shear stress not to exceed the soil shear strength might not have to be reviewed in the light of shear reversal results. So far, however, the present methods appeared to be achieving the desired benefits. Obviously drainage was of the highest importance, and there a compromise was necessary between the grading of the ballast best for drainage and the density to which it could be compacted.

It seemed to him that measured packing was logical since it involved no disturbance of a

well compacted road bed and merely tended to increase its density, but it did tend to clog up the voids on which reliance was placed for drainage. The Author's film had shown clearly that ballast deposited loosely contained a greater-than-optimum percentage of voids, of which at least some would ultimately have to be filled with stone to produce a reasonably stable ballast bed.

Mr Toms felt strongly that it was essential to have a thoroughly compacted road-bed base layer on clay or silt formations. After the use of such tools as the vibrators which Mr Alley had mentioned, Mr Toms had intended to refer to the German methods of rolling, tamping, and vibrating the sand and ballast of track foundations. He had had tests made with Vibrosoil compactors for the consolidation of Meldon dust for the bottom layer of track blankets. Although the tests had been somewhat impromptu he was convinced that in the use of compacting tools lay the biggest potential advance in foundation technique. It was no use waiting for traffic to do slowly and imperfectly the job which ought to have been done at the outset. If the consolidation could be done when it ought to be done—namely when laying the track—the speed restrictions and subsequent maintenance problems would be very much reduced. He would like to see extensive tests undertaken with the various mechanical tools which were available, including the biggest 2-ton vibrators and the lighter ones developed since, which seemed to have higher impact per unit weight.

Whether the ballast grading now accepted as standard was the best was something which he had had to consider a long time ago in connexion with the work of the Development Sub-Committee. Unfortunately, the work of that committee had not progressed far enough before it had been disbanded. A major problem was to find the optimum grading of track ballast and to study its behaviour under pulsating load. There was the frictional interlock of the stones and their elastic deformation at points of contact with one another. Under fluctuating load there must be some attrition and, as that went on, gradually the ballast broke down. Frost and weathering, of course, had a destructive effect. It might be possible to evolve theoretically an optimum grading giving the desirable properties, but he thought that it would always be necessary to bear in mind the great fund of practical experience which the railways already possessed, and no research worker could possibly ignore it.

The Author had rightly drawn attention to the effects of frost. Mr Toms quoted a case of an ash-blanketed track at Ditchling, which had become completely frost-bound. The result on thawing had been voids of 1 in. or more under the sleepers at irregular intervals along the track. A severe speed restriction had had to be imposed during the thaw. Undoubtedly drainage and the elimination of capillary-size pores for a considerable depth in the ballast layer were very important.

Mr Keith Brinsmead (Assistant Civil Engineer (Permanent Way), British Transport Commission), commenting on the fact that the Author had given as one of the functions of ballast that of resisting vertical movement of the track, suggested that there ought to be a better target than that, namely to absorb vertical movement. One of the troubles which was encountered, he said, was an attempt to resist violently the things which were happening. Two extremes were found on tracks of which he had experience. First, on the London Transport sub-surface lines the ballast had, over the years, collected a considerable amount of iron dust from the brakes and was so solid that it sometimes required the use of a concrete breaker when re-ballasting. The result of that rigidity was severe rail corrugation trouble. The other extreme was found in the Fen country, where the tracks were on ash ballast because they were always having to be lifted as the peat shrunk, where literally the track components were never worn out by mechanical abrasion.

Over the past few years the Belgians had been carrying out a number of investigations into rail corrugation and appeared very definitely to be tying it down to what was or was not the resilience of the track. An interesting fact had come out of those investigations, namely that the six-foot rail tended to corrugate more easily than the cess rail. That was put down to the lesser consolidation of the cess side of the track compared with the

six-foot side. In the present Paper, if the behaviour of the cess rail and the six-foot rail as recorded were compared, it would be found that on the cess rail more than twice the percentage of sleepers were no better after packing than before as compared with the six-foot rail, whereas only one-third of them were worsened as compared with the six-foot rail.

Mr Brinsmead's main point was the need for resilience. He challenged very strongly the long-established practice of beating ballast underneath sleepers. It was merely an expedient and was scarcely the most scientific way of doing it. The Author had described what had happened with diesel traction in Germany and in America, where vibration settled the track down. Was there not some means by which the whole track could be vibrated down? He was not thinking of the small effect produced by, for example, a Vibromax tamper shaking a little bit of ballast, but rather of high-frequency vibration as a means of settling a complete section of newly laid track. It might be worth while trying some form of high-frequency vibrator, such as the type used to vibrate concrete, which could be inserted 18 in. deep into the ballast.

The real problem, however, was how to tackle things as they existed, and it might be that American ideas of track renewal and maintenance would provide the answer. With their "out of face" renewals, as they called them, the Americans, when they did spot sleepering, gave a 3-in. lift about every 6 years. It might be said that the frequency of bridges on British Railway systems would make this impracticable, but there were in fact many miles of track on which a 3-in. lift could be given, and it seemed to him that that was the only quick and economic means of providing the resilient type of bed which was needed to support the track correctly.

Mr Frederick Fawcett (District Engineer, London Midland Region, British Railways) said that in spite of the reference in the Paper to the depth of ballast which would be necessary under modernization he, as a maintenance engineer, would still have to maintain a considerable length of plain-line track on 7 in. of ballast, and perhaps less.

The essential features of maintaining shallow depths of ballast were screening, draining, and packing. Screening and draining could be done by almost any class of labour, if available, but maintenance hand packing required skill and experience. With the present trend of manpower in his District he imagined that in a very short time he would find neither muscles nor experienced intelligence in the District, and he would have to have a system of packing which accorded with the type of manpower at his disposal.

The Author had implied that the present methods of hand packing were failing, and Mr Fawcett agreed with him. Beater packing in Mr Fawcett's District might be useful owing to the lack of experienced intelligence, but he could not re-introduce it because he had no staff of sufficiently robust physique. He still thought, however, that a beater bar was one of the most useful tools for the packing of switch and crossing timbers but emphasized that the problem was to find a solution to the present difficulty of obtaining an effective standard of packing of plane track; and he did not agree with the Author that it lay in off-line mechanical tampers. The Author had said that the tests at Tamworth were not conclusive. That seemed to be the case with most tests which had been made with off-line tampers although Mr Alley had been a little more conclusive. The fact had to be faced that platelayers had to do the work and that led to the question of human relations. Those men could not manhandle a 3-cwt or 6-cwt compressor or prime mover up and down the cess, which in some places was very narrow and restricted. Mr Fawcett felt, therefore, that it was not possible to rely on off-line tampers to eliminate manpower troubles.

Even after more than a hundred years of railway operation there was still need of a method of packing which was within the capabilities of present-day staff. He had been interested in the Author's reference to an experiment in 1906 with the grouting of track by air-blown aggregate. Mr Fawcett had been making experiments in that direction, without success; he had been checked by the difficulty of getting compressed air where it was wanted.

In the meantime Mr Fawcett would rely upon the present-day Matisa tamper, which was doing a first-class job. He had two of them working in his district and he had been

trying hard to get more. One of them in particular was being used by a very good ganger, who was covering up to 1,000 yd of track on a Sunday, finishing with an excellent top, packing all the sleepers so well that 5 months afterwards there was very little depression and very little movement. He had found that the best results were obtained when the track was set to level, ahead of the machine, by the use of measured-shovel packing, sighting boards, and Matisa jacks giving a general lift of about ¼ in.

However, as the Author and Mr Alley had pointed out, the full value of the on-track tamping machine was not being realized because of lack of occupation facilities. He noticed in the Paper that the German machine had an output of 65½ miles per annum. The output of one of Mr Fawcett's tampers was about half that figure, but in spite of that and in spite of Mr Alley's economics Mr Fawcett felt that the Matisa was a worthwhile machine and he felt that the Operating Department should be persuaded to realize the possibilities of the Matisa machine and grant increased possessions. For example, he had four lines of way on his two trunk lines which each extended 50 miles out of London. He had asked in vain for two nights a week to put the Matisa tamper on the fast lines. On a short possession of 5 hours he could get ¼ mile of track tamped soundly and solidly.

The Author had recommended a 12-in. ballast shoulder for its value in resisting lateral displacement of the track but it was also necessary to have a heavy shoulder to support the cess rail which invariably lost elevation far quicker than the 6-ft rail.

Mr H. H. Robinson (Assistant Engineer (Permanent Way), Eastern Region, British Railways) said that the Matisa machine had been criticized in the past because of the doubtful top which it sometimes left behind. Mr Fawcett's experience with the modified method of working the Matisa machine was very interesting, because on the Eastern Region, by the adoption of the same methods, the same excellent results were being achieved. That seemed to be the answer to the question of whether or not maintenance packing could be done by the Matisa machine without a lift of the road. The answer was that it could be and was being done. By the use of Abtus siting boards and obstructionless jacks in conjunction with the Matisa machine, a top of the order of 88–90 was being achieved using normal standards of marking.

It seemed to him that the Author had done rather less than justice to measured shovel-packing. The difficulties were known: it was agreed that the method was initially slow and that it required care; in some places the necessary manpower was not readily available. Nevertheless, the fact was that in proper conditions—conditions which were adequate, but not necessarily perfect—it produced a first-class job and that a first-class top remained under heavy steam traffic for a long time.

Mr Robinson was also interested in the historical notes in the Paper. It was interesting to note the very wide divergencies in practice after more than 75 years of railway maintenance. In 1900 three of the major companies still used ballast carried up to the rail head over the ends of the sleepers, and one of them used it in the four-foot as well. One of them, surprisingly, omitted the outside ballast shoulder entirely. All that seemed to show that even by 1900 the true functions of ballast had not been understood.

In Appendix I the Author, referring to the Bosch tamper, had stated that the weight of the blow was adjusted so as not to break the ballast. Was that true of slag ballast which was used in places on the L.M.R and had any trouble been experienced in that respect?

On p. 187 the Author had quoted the cost of Matisa tamping per mile (and presumably per annum) as £230. That was extremely high compared with Eastern Region practice. Had the track required tamping more than once per annum with the Matisa tamper, and how large a gang had been required to operate it?

On the Eastern Region it had been realized that it was necessary to consolidate the ballast to a depth of 9 in., which is greater than that which the Matisa could reach (3 in.). Trials had been made with the Vibromax consolidator and with the Vibrosoil consolidator, and the results seemed to be promising; the experiments were being continued. It was to be hoped, as Mr Toms had suggested, that some really worthwhile tests could be made and reliable results achieved.

Mr Sidney Stevens (District Engineer (London), Western Region, British Railways) agreed entirely with Mr Fawcett's down-to-earth remarks. Whatever system of consolidating ballast was adopted it was the man on the job who mattered.

Bearing that in mind Mr Stevens proposed to deal primarily with portable tampers. A portable tamper, or a portable machine of any kind whatever, which had to be used in the open in all weather conditions and by men of varying intelligence and physique, had to fulfil two conditions: it had to be light in weight, and it had to afford a comfortable operating position for the man using the tool. If the latter function were not given proper consideration the tool would not be used over sustained periods every day of the year, or at varying times of the year over a number of years. Tests over a short period, as the Author had said, would be inconclusive. Mr Stevens gathered from the Paper that the tests which the Author had described consisted of one day on plain line and part of a day on crossing work, and no doubt the tests were carried out with very senior staff looking on. When a man was asked whether he liked a machine his answer might depend on his attitude towards his "boss." It was an entirely different matter when the man had to use the machine all the year round.

From that point of view Mr Stevens had personally operated the Vibromax, the Kango, and the Bosch machines. The Kango was only about one-third the weight of the others; it was a streamline design, easy to handle, and fell naturally into an easy operating position. The Bosch required an additional strut to take part of the load, which hampered the working of the machine when it was being manœuvred by a lengthman.

The Author's comment in Appendix I that "No sense of vibration is conveyed through the handle to the operator" could well be applied to the Kango hammer.

Coming to the question of testing under practical conditions, he thought that all such tools and machines required at least 12 months in actual service before it was possible to say "This is a good machine; we can use it, and over a long period it will be economic."

Early in 1947 the London Division of the Great Western Railway had been looking for a mechanical portable tamper and had adapted the commercial Kango hammer. They had had to wait for the development of a larger design of tool, the E-type hammer, which had then been in the design stage, and was then considered for ballast tamping in addition to building work. They had carried out tests in 1948, and as a result of the tests at Paddington and the improved results obtained with crossing work the adoption of the Kango had been agreed for point-and-crossing work only. It had been obvious from the beginning that it was too slow for plain line work.

Kango hammers had been in use on the Great Western and afterwards on the Western region since 1948. The spread of them had been rather slow because of supply difficulties for materials and for tools, but he proposed to give a few figures to show what could be done when they were properly and regularly programmed in their use. He emphasized "programmed," because with all portable tools it was necessary to programme their use if too high a proportion of idle time was to be avoided. On the Didcot Section of the London District there was a main-line section with high-speed track carrying such trains as "The Bristolian" and "The Pembroke Coast Express," both of which would be travelling at about 80–85 m.p.h., both Up and Down. The Kango tampers had been in regular programmed use for the past 3 years. It had been shown—and it applied to all tamping methods and ballast consolidation in crossing work—that it was not possible to segregate one part of the job from another. Tamping had to be properly co-ordinated with fettling the components and with welding the crossings because, however well an appliance would consolidate the ballast under a crossing, if the components were in a poor state of repair the ballast consolidation would deteriorate rapidly and the tamping tool would carry the blame.

In order to see what could be done with combined maintenance of the crossing work, of which tamping was a very big part, for the past year a programme of systematic maintenance of the crossing work had been carried out regularly by the Inspector on the Didcot Section. On an average the normal day-to-day methods of maintaining crossing work had involved roughly 8 man-hours per week on fitting work approximately 3 years old. That

time increased with the age of the fitting, because it was not possible to pack crossing work effectively with a shovel and the rate of deterioration increased with age, which involved expenditure on replacing fastenings, crossing timbers, and component parts.

With the systematic maintenance there were three main stages. There was first of all the fettling of the component parts, which included tightening of fastenings—he referred at the moment to bullhead track—changing chair-worn and damaged timbers, particularly under the crossing nose, and fettling thoroughly, following that immediately with welding of the crossing vee and wings or elbow points and back rails to eliminate the accentuation of hammer-blow at the gap, and then tamping the fitting with the Kango hammers, using two hammers opposite each other, run off a 1½-kW portable generator set. The number of men used with the Kangos was five, including the lookout man. The tamping for a normal fitting, of 1 in 9 or 1 in 10 angle, was 20 man-hours. The actual welding depended on occupation, but if done in one occupation it took about 8 hours and if done between trains it took 3 to 4 days; but it was programmed systematically, taking into account all the conditions.

With the fitting work dealt with in that way, fittings which had taken something of the order of 8 man-hours per week now stood up to main-line fast traffic and did not require attention for periods of from 3 to 4 months, and in some cases 6 months, and their running was perfectly good.

Those figures showed the general advantages which could be obtained from combining the crossing-work consolidation of ballast with the other essential features of maintaining that type of feature of the track. The men soon got into the habit of using the Kango, which was easy to handle, as no doubt the tests had shown. The generating unit was a two-man job. The auto-diesel, which was a small machine mounted on caterpillar tracks, was very manœuvrable and could be easily accommodated in normal cesses. As he had already said, five men were required. They had tried using the large generator and four hammers, but this took men from the length gang, and a length ganger was inclined to hurry the work because he had no men for other jobs, whereas if two men were put on the site and three taken from the gang the ganger had another man or two to do other work. In any case, it was the minimum number for the normal continual work which was necessary to achieve the object desired.

On the question of frost protection, the Author had mentioned a minimum of 15-in. depth of ballast. Presumably that meant 15 in. below the underside of the sleeper, and was not measured from its top.

The Author, in reply, said that Mr Stevens had at first remarked that he could not see what use line-side tampers were going to be, and had then extolled the virtues of the Kango. Whilst holding no brief for any particular unit, the Author would suggest that each of them showed sufficient promise to warrant further interest being taken in it and further experiments being conducted with all of them. Naturally one demonstration, which might perhaps be conducted under abnormal working conditions, would not give the results obtained over a long period, but it was not possible to say "The men will not like them" and leave it at that. To say that might be one reason why something was wrong with the railways today.

Mr Fawcett had mentioned that with his unskilled labour, having neither muscle nor brain, he was having to rely on a machine to do the job, and he wanted further possessions to allow that machine to do the job. It would be very nice to have further possessions, but it was necessary to consider the people who used the railways; the railways were not there simply for District Engineers to play about with.

Naturally trials of equipment would have to take place over a considerable period, and no spot conclusions could be formed. With regard to the depth of ballast, railways had been in existence for a considerable time and it was still necessary to find the answer to that question. He suggested that engineers should get together and produce an answer. Mr Toms, supporting that, went further and would like additional experiments to take place on formation in relation to depth of ballast.

Mr Brinsmead had raised the interesting question of the high-frequency vibrating of long lengths of track. Two lengths of track in Switzerland had been treated in that way, but the power output required to produce the vibration had naturally, at the present stage of development, not been economic.

The question had been raised of the effect of percussion tools on slag ballast. There were very many opinions about slag ballast, and if a machine which vibrated ballast could smash slag ballast, as had happened on one Region recently, the Author would suggest that there was something wrong with the slag ballast rather than with the machine. It was interesting to know that the Eastern Region were experimenting with the surface vibrator, the Vibromax 500, for consolidating. Probably quite useful results would be obtained. Perhaps immediately following re-laying, or at least before the new track was put back, and even perhaps before ballast could be put down and consolidated to a predetermined level. There again continuous experiment would have to take place over a period.

ROAD ENGINEERING DIVISION MEETING

24 January, 1956

Mr Arthur Floyd, Member, Chairman of the Division, in the Chair

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Division were accorded to the Author.

Road Paper No. 50

THE OXTON BY-PASS EXTENSION

by

*** Reginald Alexander Kidd, B.Sc., M.I.C.E.**

SYNOPSIS

There has been little experience of the construction of concrete roads in recent years. Nevertheless, views on design and construction have advanced. A description is given of the work on the extension of the Oxton by-pass, designed in the light of present-day knowledge and undertaken with plant previously used on airfield construction and adapted for road construction. The length of the road is only ¾ mile, but economic considerations were sacrificed to some extent in the desire to establish the machines required and the technique of construction considered suitable for the construction of a 10-mile section of road.

The subgrade was prepared prior to the main contract, and particular attention was paid to this with a view to achieving a foundation which would have a good uniform bearing value with adequate drainage and surface sealing to avoid any large variations in moisture content.

In the construction of the road the objects aimed at were to lay a concrete carriageway of high flexural strength, with a running surface as near perfection as possible, which would stand up to heavy traffic with minimum maintenance and freedom from cracking, and which would offer good resistance to skidding.

The concrete mix to achieve the specified minimum flexural and compressive strengths, with variations during construction, together with the set-up of batching plant, concreting machinery, and incidental equipment, are described.

The design provided for expansion joints only at 95-ft and 113-ft spacing. These were the cause of considerable surface irregularity, and although this was largely overcome on the running joints the problems associated with those constructed at the end of a day's work were not solved.

Tests carried out during and after the work are described. These show that the strength of the concrete, as specified, was achieved in 82% of the specimens. Riding qualities fell somewhat short of the high standard aimed at; the chief reasons for this are thought to be formwork, joints, and lack of continuity of the concreting operations. Resistance to skidding is considered to be good.

The experience gained from this work is regarded as being of great value in establishing standards for design and methods of construction of future major road works.

Introduction

Since the early days of the 1939–45 war very little construction has been undertaken in concrete on major roads in Great Britain and indeed very little machine-laid

* The Author is County Surveyor of Nottinghamshire.

concrete exists on such roads. Many road engineers are still, however, anticipating the time when large-scale road construction may start and it is desirable from time to time to take stock of the knowledge and ability available to undertake extensive concrete construction, should the opportunity occur.

In the meantime, a great deal of concrete has been laid by mechanized methods in airfield construction. British contractors have acquired plant for such use and have developed specialized construction techniques. Certain European countries have continued their pre-war programme of concrete motor-road construction and many British engineers have examined and carefully studied their methods and standards.

Turning to the past history of concrete-road design in Britain there is no gainsaying that the basis of design, so far as thickness of slab, weight, and position of reinforcement, etc., were concerned, was by rule of thumb; certain popular views on such details had been established and were followed without any real scientific basis. Unfortunately many failures occurred, but the work of the Road Research Laboratory in studying such failures has been of inestimable value, and three principal points have emerged therefrom. First, the very general lack of consideration to proper preparation of the subgrade to receive a concrete slab, arising undoubtedly from an erroneous belief that the concrete was a raft which would bridge over all weak places in the formation. Secondly, faulty construction and spacing of joints resulting in bad riding qualities, later aggravated by cracking and relative settlement of slabs. Thirdly, the need for proper and economic use of reinforcement so that this bears a definite relation to the slab length, it being now more fully realized that the shrinkage and temperature stresses can be of a much higher order than those induced by slab loading.

What of the future of concrete-road construction? One point to which considerable regard must be paid is that there are other forms of construction which may be competitive from an economic point of view and can undoubtedly be competitive in regard to good riding qualities, in fact the black-top surface laid by machine sets the standard of good riding in Britain. The virtues of concrete as compared with alternative forms of construction will therefore have to be assessed very carefully in considering any large-scale roadworks in Britain.

The present-day conception of the problem connected with concrete-road construction can be considered in relation to certain basic characteristics, namely:—

(1) Initial cost.
(2) Economic life.
(3) Extent and cost of the maintenance required.
(4) Riding qualities of the road at the time of construction and throughout its economic life.

The aim must be to construct and maintain the road for a period of, say, 40 years at a cost which would be no greater than that of other forms of construction with riding qualities which would be equally as good when constructed, and which would remain in better condition throughout its life with a negligible amount of maintenance.

As a result of work undertaken by the Road Research Laboratory and experience of engineers in other countries a new understanding of the value of the material underlying a concrete road has emerged, whether such material be in the form of natural soil trimmed off to form the subgrade or in the nature of an imported and consolidated sub-base. It has been established that the soil should possess uniform bearing value over the whole area and in the case of the Oxton by-pass extension this was achieved by adding a base of well-consolidated burnt colliery shale which was

sealed with a surface dressing, as described later. Even though the bearing value may be fairly low, if uniform moisture conditions are maintained in this way the concrete slab may be expected to carry traffic successfully for a long period.

Following a visit to Belgium and Germany when discussions with German engineers on the design and technique of construction of concrete roads took place, the idea was formulated of applying the available knowledge of the factors mentioned above, affecting the construction and life of a concrete road, to a scheme for the extension of the Oxton by-pass in Nottinghamshire. The aim was to ascertain how high a standard of construction could be achieved, with particular emphasis on riding qualities, soundness of construction, practical considerations in forming joints, and the most satisfactory ways of using the large plant now normally engaged on airfield construction.

Location of works

The Oxton by-pass is a link in the class 1 (A.6097) road which acts as the outer by-pass of Nottingham, for traffic proceeding from the neighbourhood of Leicester *via* the Fosse Way (A.46), Gunthorpe Bridge and A.614, to Ollerton and the north. In this respect it takes a considerable proportion of commercial traffic and is one of the main routes for heavy out-of-gauge loads between South Wales, Rugby, and the north.

The traffic census taken in August 1954 indicates that a total of 24,721 vehicles per week use this road, of which 4,944 are heavy goods vehicles.

Since the length of road available was only $\frac{3}{4}$ mile, there was a definite limit to the experience that could be obtained in regard to first cost of construction, and economic considerations were sacrificed in order to determine the most suitable technique and machinery for the construction of a considerable length of concrete road with a high-quality finish. The plant actually used in the construction was that considered suitable for the construction of a 10-mile section of concrete road.

The section of road in question forms the southern end of the by-pass scheme and land acquisition with some earthworks in cut and fill were undertaken just prior to, or in the early days of, the 1939–45 war. The northern end of this by-pass is in dual-carriage way construction, one carriageway having been completed in 1940 in hand-laid-concrete alternate-bay construction. The other carriageway was constructed in part as an experimental road in 1947, the performance of which was described by Loe.[1] The location is shown in Fig. 1.

The work described on the extension of the by-pass is thus close to other examples of concrete-road construction, so that immediate comparisons are available. The details and comments that follow are all related entirely to the works carried out in 1955 and known as the Oxton by-pass extension.

Date of construction and weather conditions

The preparation of the road formation was put in hand during the winter of 1954–55, the final shaping and sealing with tar being undertaken during June–July, 1955. The first stage of the concrete carriageway consisted of laying a trial bay 200 ft long, which was constructed on the 23rd, 24th, and 25th August. There was then a lapse of 18 days during which time test cores were taken and tested; also the contractor had an opportunity to make any minor alterations and adjustments to his plant.

[1] J. A. Loe, "The Performance during the First Five Years of the Experimental Concrete Road at Oxton, Nottinghamshire." Proc. Instn Civ. Engrs, Pt II, vol. 4, p. 137 (Feb. 1955).

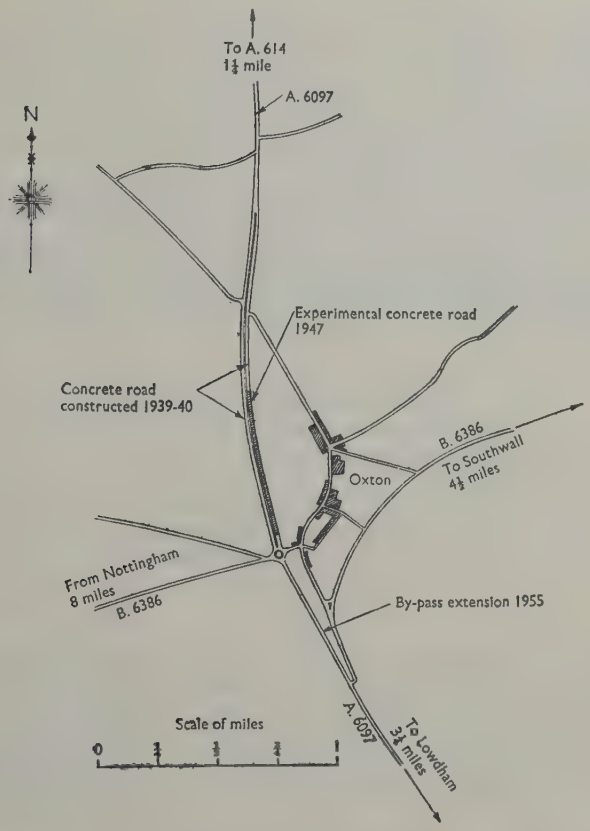

FIG. 1.—MAP SHOWING LOCATION OF SITES

Concreting on the main slab was commenced on the 12th September in hot summer weather and continued until the 5th October with dry weather throughout, but becoming much cooler after the first week.

ROAD LAYOUT

The width of formation already roughly prepared permitted a layout as follows, having some features of continental practice:—

Length	3,625 ft
Carriageway	22 ft wide; white edging strips (each 12 in. wide); flush hard shoulders (each 5 ft wide); a total formation width of 34 ft
Length of bays	Generally 97 ft and 113 ft
Thickness of concrete	8 in.
Crossfall on carriageway	1 in 48

Preparation of formation

The subsoil throughout the section varies considerably, from fine sand overlying the Bunter Sandstone to clay with pockets of fine silt. The former has a good bearing value but the latter a very poor bearing value, especially when wet.

The design provided for adequate subsoil drainage and the provision of a waterproof layer over the whole formation width with a view to the maintenance of uniform moisture content in the subgrade.

Land drains (4-in. dia.) surrounded with gravel were laid at a depth of 2 ft below the finished surface, 6 ft 6 in. outside each edge of the carriageway. The water from these land drains was collected into small chambers at frequent intervals and conveyed into open ditches constructed near each highway boundary. Surface water from the road over the hard shoulder either soaks away direct into the land drains or is collected by means of gulley gratings fixed over the small chambers.

Table 1.—Bearing values of subgrade and base

Test site No.	27th April, 1955		11th July, 1955		Remarks
	C.B.R. value: %	Moisture content: %	C.B.R. value: %	Moisture content: %	
1	27	—	—	—	On 17 in. of burnt-shale base
(north-west end)	30	15	—	—	On underlying subgrade of sandy gravel with brown clay
2	59	—	—	—	On 12 in. of burnt-shale base
	6	15½	—	—	On underlying subgrade of sandy gravel with brown clay
3	43	—	57½	—	On 12 in. of burnt-shale base
	1½	19½	7	13	On underlying subgrade of brown silty clay
4	29	—	42	—	On 14 in. of burnt-shale base
	2½	18½	6½	14	On underlying subgrade of brown clay with pockets of silt
5	8	—	16	—	On 5 in. of burnt-shale base
	3	26	6	16½	On underlying subgrade of yellow-brown silty clay
6	29	—	32½	—	On 14 in. of burnt-shale base
(south-east end)	6½	13	8½	11	On underlying subgrade of brown silty clay

The remaining earthworks on the section were carried out during the winter of 1954–55, when owing to the very wet weather the silty clay became quite unsuitable as filling material and had to be bulldozed out on to the margins. Filling under the carriageway was then carried out using burnt colliery shale, which was available within 5 miles of the site; a minimum depth of 6 in. of this material was aimed at, although in many places the thickness was very much in excess of this.

The shale was compacted with a 30-cwt vibrating roller and after the subsoil had dried to some extent the formation was finally shaped up to correct levels, twice tar-sprayed, and blinded with $\frac{3}{16}$-in. gravel to the full formation width of carriageway and hard shoulders, namely, 34 ft. The bearing values above and below the shale are indicated in Table 1, and it will be noted that when the tar-spraying was carried out during July the bearing values of the subsoil were more uniform and higher than in April. As would be expected, the bearing value of the shale base improved slightly but is good in both cases.

The high C.B.R. values of the shale are worthy of note but there is a tendency for this material, unless well burnt, to revert to clay under the action of water, and adequate sealing is very necessary. An area in the vicinity of test site No. 5 was taken out and replaced immediately prior to concreting. It is clear that an average C.B.R. value of 35% to 40% was obtained and that the minimum bearing value of the base exceeded 20%. These results are interesting in that they confirm opinions as to the advantage of the granular base and the surface dressing in improving the uniformity and strength of the foundation.

Riding qualities

As has already been indicated, particular emphasis was given to the endeavour to produce a very high standard of riding qualities and in this connexion regard was had to the factors affecting the quality of machine-laid concrete roads described by Kirkham.[2] These factors are spreading, finishing, form work, and joints; a great deal of consideration was given to these factors both in the design of the work and during construction. The points that emerged will be dealt with later.

Design of concrete slab

A formation having been achieved which ultimately carried constructional traffic without distortion, the next object was to construct a concrete road with the following characteristics:—

(1) High flexural strength.
(2) A running surface as nearly perfect as possible.
(3) A running surface which would stand up to heavy traffic with a minimum of maintenance and freedom from cracking for a period of at least 20 and possibly 30 or 40 years.
(4) A running surface which would offer good resistance to skidding.

The specification set very high limits of 5,000 lb/sq. in. compressive strength and 600 lb/sq. in. flexural strength, both at 28 days. These values were chosen as a result of tests on trial mixes using the aggregates proposed for the work and it was realized that they would be produced on the job only by very careful attention to the

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[2] R. H. H. Kirkham, "Factors Affecting the Riding Quality of Machine-Laid Concrete Roads." Proc. Instn Civ. Engrs, Pt II, vol. 2, p. 656 (Oct. 1953).

design of the mix and by the exercise of adequate controls. So far as is known, concrete of such high strength has not previously been specified for road work in Britain. The specification also required that full compaction should be produced by compacting in a single layer and that the contractor should aim at producing a riding surface having an irregularity index q as measured by the Road Research Laboratory's multi-wheeled profilometer, of not more than 40 in/mile.

The anticipated intensity of traffic on the road was "medium to heavy" within the "Classification of Traffic—Table I" of the Road Research Laboratory's Road Note No. 19, and the slab thickness selected was consequently 8 in. with 10 lb/sq. yd of reinforcement (Road Note No. 19, Table II—Normal Sub-grades). It was decided at Oxton to omit contraction joints and to place expansion joints as infrequently as possible. In all, thirty-three dowelled expansion joints were formed; of these sixteen were formed during a day's work and seventeen at the end of a day's work, concreting always terminating at an expansion joint. The spacing actually adopted was 97 ft and 113 ft, these being multiples of the 17-ft lengths of the reinforcing sheets, allowing 12-in. lap. To overcome difficulties at a farm access, however, one bay was made 145 ft in length.

Full-width construction of the concrete slab was favoured at the design stage for two main reasons:—

(*a*) To enable progressive forward movement of the machines in the interest of good practice.

(*b*) To eliminate some of the possible causes of irregularity of surface.

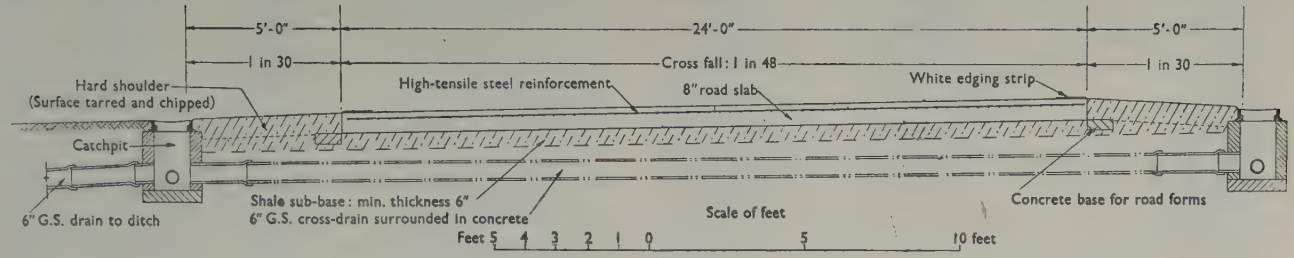
Tenders were, however, invited on two alternative bases, namely, full-width and half-width slab respectively; the price for the full-width slab was in fact the lower. This introduced a third and very convincing reason in favour of full-width concreting and the final decision was made accordingly.

The initial strains due to shrinkage and under-surface drag were taken care of by providing top single-layer reinforcement as described below, laid $2\frac{1}{2}$ in. from the top of the slab; in the case of the centre mat, the extra transverse steel acted as a tie across the sawn centre joint:—

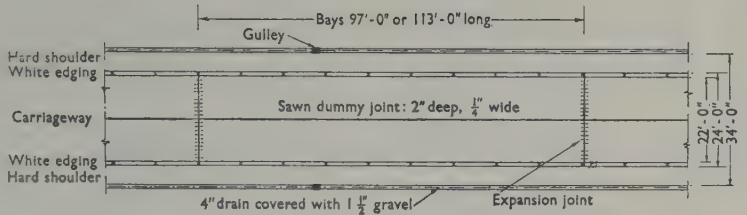
Outer mats	Longitudinal wires 2/0 gauge (0·348-in. dia.) at 4-in. centres. Transverse wires 4 gauge (0·232-in. dia.) at 16-in. centres. Weight 9·69 lb/sq. yd
Middle mat	Longitudinal wires 2/0 gauge (0·348-in. dia.) at 4-in. centres. Transverse wires 1 gauge (0·300-in. dia.) at 8-in. centres. Weight 11·96 lb/sq. yd

Both these fabrics were specially made for this job with wider spacing between longitudinal wires than in the standard fabrics made by the Reinforcement Manufacturers' Association. It was desired to ensure full and complete compaction of the concrete in single-layer construction without any possible adverse effect arising from the presence of the reinforcement mat.

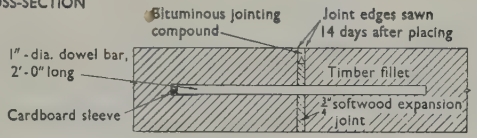
A cross-section of the concrete slab and hard shoulders is shown in Fig. 2a.



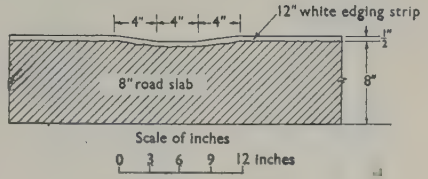
(a) TYPICAL CROSS-SECTION



(b) PLAN


Bituminous jointing compound
Joint edges sawn 14 days after placing
1" -dia. dowel bar, 2'-0" long
Timber fillet
Cardboard sleeve
1/4" softwood expansion joint

(c) DETAIL OF EXPANSION JOINT



(d) DETAIL OF EDGING STRIP

FIG. 2.—ROAD-SLAB DETAILS

Concrete mix

The cement used was ordinary Portland cement, the coarse aggregate a crushed Leicestershire granite supplied in two single sizes, namely, $\frac{3}{4}$ in. and $\frac{3}{8}$ in., and the sand one from the Trent Valley with a grading falling within zone 3 of B.S.882 : 1954. The mix proportions of the concrete for the trial slabs and for the first slab of the road proper were 1:6·5.

Cement	1 part
Sand	1·95 parts (30%)
$\frac{3}{8}$-in. crushed granite . . .	1·55 parts (24%)
$\frac{3}{4}$-in. crushed granite . . .	3·00 parts (46%)
Water/cement ratio . . .	0·55

With this mix some difficulty was encountered in obtaining a satisfactory surface finish and for the remainder of the road (with one minor exception) the mix proportions was reduced to 1:6·25 and the proportions of the aggregates adjusted as follows:—

Sand	32%
$\frac{3}{8}$-in. crushed granite . . .	18%
$\frac{3}{4}$-in. crushed granite . . .	50%

At the same time the water/cement ratio was increased to 0·58. This mix was used for slabs 2 to 10. For slabs 11 and 12 the water/cement ratio was reduced to 0·57 and for slabs 13 to 17 to 0·56.

During the work on slab 18 the proportions were further changed to:—

Sand	30%
$\frac{3}{8}$-in. crushed granite . . .	18%
$\frac{3}{4}$-in. crushed granite . . .	52%

and the water/cement ratio reduced to the original figure of 0·55. The object of this change was a reduction in the amount of surface laitance, which had tended to be excessive. The change produced difficulties in obtaining a sealed surface and from slab 19 onwards the mix reverted to that used immediately before the change. The water/cement ratio, as measured by the quantity of water in the sand and that added at the mixer to give a suitable workability, subsequently varied between 0·55 and 0·57.

Plant set up

Batching plant

Two twin-compartment weigh-batchers (only three of the four compartments available were used). These were fed by grab from four stockpiles; one of $\frac{3}{4}$ in. single-size aggregate (to B.S.882), one of $\frac{3}{8}$ in. single-size aggregate (to B.S.882), and two of sand (for use on alternative days).

Cement handling plant consisting of elevator belt and two 5-bag hoppers. (Although bulk cement would normally be used for a large road contract, the circumstances applying at Oxton were considered to justify the omission of a large silo and handling plant matching the output of the weigh-batching equipment.)

Transport of dry batches

Four 5-ton tipping lorries fitted with twin compartments for holding two dry batches.

Mixing

One twin-drum paver with 1,000-gal water-tanker for supply of mixing water. (The water-tanker was also used for carrying water to wash down concreting equipment after the completion of the day's work and later to supply water for concrete-joint sawing equipment.)

Spreading

One 20-ft paddle spreader modified by widening to 24 ft and by the substitution of a spreading screw for the paddle.

Finishing

One 24-ft vibrating finisher with smoothing beam. The spreader and finisher followed by the wet-surface profilometer are shown in Fig. 3, facing p. 218; the wet-surface profilometer is described later.

Joint cutting

One industrial abrasive electrically driven saw with either diamond or carborundum saw discs together with generating set and vacuum pump.

One petrol engine driven saw with either diamond or carborundum discs.

Incidental equipment

A smooth-wheel roller for compacting make-up gravel on formation. Mechanical barrows for transporting concrete or mortar used for bedding road forms and for white edging strip.

Small 5/3½ mixer for white concrete.

Vacuum pump for the suction pads holding down the guide for the concrete saws.

Control of concrete quality

The quality of the concrete was controlled by a concrete engineer appointed by the contractor. The moisture content of the sand was measured each day before work started and due allowance made to the water added at the mixer. Since weather conditions were favourable for the duration of the job, the moisture content of the coarse aggregate was negligible throughout. The maintenance of large stockpiles of aggregates assisted in maintaining uniform moisture conditions.

The workability of the concrete was checked periodically by measuring the compaction factor and the general range of variation was from 0·81 to 0·85.

Four 28-in. × 6-in. × 6-in. beams were made during each day's concreting. These were tested at the Department of Civil Engineering of the University of Nottingham at the age of 28 days for flexural strength and for crushing strength as measured by the equivalent-cube test. The tests were made in accordance with B.S.1881 : 1952. The results are shown in Table 2.

Trial slab

Eighteen days before work started on the concrete carriageway proper, a length of about 200 ft was laid to try out the plant and methods to be used. It was finished 3 in. below running level and will be surfaced with a bituminous carpet to match the existing flexible pavement at the south end of the scheme. Cores were cut to determine the degree of compaction obtained and the location and degree of bond of the reinforcement. The trial resulted in the introduction of several improvements and familiarized the contractor's staff with the relatively novel constructional techniques being used.

TABLE 2.—TESTS ON BEAMS

Beam Nos	Average compressive stress: lb/sq. in.	Average modulus of rupture: lb/sq. in.	Beam Nos	Average compressive stress: lb/sq. in.	Average modulus of rupture: lb/sq. in.
7 & 8	5,785	645	39 & 40	5,020	620
9	5,350	590	41 & 42	5,690	635
11 & 12	5,458	645	43 & 44	5,578	675
13 & 14	5,253	670	45 & 46	6,150	690
15 & 16	5,793	680	47 & 48	5,728	715
17 & 18	4,950	565	49 & 50	5,395	655
19 & 20	5,913	675	51 & 52	5,300	760
21 & 22	4,668	595	53 & 54	4,448	605
23 & 24	4,935	595	55 & 56	5,813	625
25 & 26	6,093	775	57 & 58	5,583	620
27 & 28	5,425	740	59 & 60	5,168	655
29 & 30	4,428	575	61 & 62	5,310	660
31 & 32	4,593	685	63 & 64	7,330	795
33 & 34	3,898	555	65 & 66	5,235	610
35 & 36	7,588	785	67 & 68	6,843	750
37 & 38	7,290	775			
		Standard deviations		847	69

Figures relate to average readings for each pair of beams

BATCHING AND MIXING

Reference has already been made to the important requisite that the plant provided should be that appropriate to a large road-paving contract involving the construction of up to 10 miles of dual-carriageway road.

Generally speaking there are two methods available for the supply of concrete to the work:—

(*a*) The use of a central batching and mixing plant, the mixed concrete being carried to the point of deposit in lorries or dumpers.

(*b*) The use of a central batching plant, measuring out dry batches which are carried by lorries or other means to a mobile mixer at the working place.

The contractor selected to carry out the work at Oxton had a good deal of experience of method (*b*) and considered that the closest possible control over concrete quality—particularly as regards the maintaining of constant workability at the point of deposit—which would be required if a uniform riding surface were to be attained, would most easily be achieved by this method. The contractor had under development a new type of spreader which he considered would achieve greater uniformity in the density of spread concrete than had proved possible with the types available. At the commencement of the work the development of this spreader had not reached a stage where it could accept concrete tipped from lorries, but could readily handle and spread concrete deposited in the centre of the spreader by means of the boom-and-bucket device in a mobile mixer of the paver type (Fig. 4, facing p. 218).

The central batching plant was erected on existing hard standings approximately 1¼-mile north of the southern end of the work and about ½-mile north of the northern end.

The layout of the site imposed certain restrictions on the use of a mobile mixer operating outside the formwork of the carriageway under construction and, in view of the fact that the prepared formation had a high bearing value, it was considered expedient to operate this item of plant, together with its attendant water tanker, on the road formation. Since, along the major portion of the road, the mixer was operated between the road forms this implied that the lorries carrying dry batches should also be allowed to travel on the formation.

The performance of the formation during the construction of the concrete slab indicated that agreement to this concession was justifiable, there being very little deterioration of the formation under the heavy loading conditions imposed.

Although this expedient proved generally satisfactory at Oxton, it was noted that occasionally the water used to wash down plant at the end of a concreting session or spilled during the transfer from water-tanker to mixer tended to cause some softening of the formation and to affect the workability of future concrete laid over the wet patch. The necessity of washing down spreading and finishing plant with water which ultimately collects on the formation presents a real problem; water from spillage during the charging of the mixer can be avoided by operating the mixer outside the forms. Where site conditions allowed, this was in fact done.

The mobile mixer used was capable of efficiently mixing a batch of 37½ cu. ft of concrete (measured in its compacted state), but because of limitations imposed by the experimental nature of the spreading plant the batch size was limited to 27 cu. ft (approximately). Total mixing time in both drums of the paver was 2 min and the concrete was fed to the spreader by the boom-and-bucket attachment of the machine.

Forms and form laying

It is axiomatic that where the profile of a surface is obtained by screeding transversely between forms, then the profile of the finished surface is in large measure dictated by the accuracy of the formwork. For this reason it is essential that when an accurate surface profile is required the formwork must have a true screeding edge and must be fixed accurately to correct levels.

Heavy duty road-forms of 7½-in. overall depth and 10-in. base width were used throughout the work. The forms were provided with independent rails to length of 10 ft, the rails being carried on the stake housings. Any forms which showed a departure from a true straight edge of more than ⅛ in. at any point were rejected.

All formwork was carried on a concrete beam 4 in. deep laid outside the slab proper and each form was bedded on lean mortar ½ in. to 1 in. thick to bring it up to true level. The level of each form was determined from pins, driven into the formation at 10-ft intervals adjacent to each end of the form and set by means of a precise level, a 2-ft spirit level spanning from pin to form.

Formwork was secured to the concrete beam by means of coach screws, the anchorage for the coach screws being asbestos filler in drilled holes.

The major fault in the formwork was the dip between the end of the forms and the first stake housing, apparently produced in manufacture. A suggestion was put forward that to overcome this difficulty the rail itself should be made the screeding edge for the concrete and the true level maintained by shimming up the rails as required. This expedient was considered impracticable on the following grounds :

(1) The rails could not be guaranteed to be of uniform depth.
(2) Any loose shims or packing might easily be displaced during the passage of the vibrating machine.

(3) Shimming might be required at several points along the length of a rail unless the setting and welding of the stake housings were very accurately carried out.

(4) The majority of the forms were not adaptable to the use of shims.

The contractors spared no effort to obtain the maximum practical accuracy in form setting, but the initial inaccuracies in the forms themselves resulted in a profile along the forms which was rather below the standard originally aimed at. Progress in form laying was generally slower than that of concreting and on a number of occasions work was delayed owing to an insufficient length of formwork being available. This led to working over forms laid only the previous day, but the mortar bedding to the forms had no doubt developed sufficient strength to carry the load without distortion. When the trial slab was laid levels taken before and after the passage of the machine showed that the forms remained rigid and did not move or distort.

Placing of reinforcement

The specification permitted two-layer spreading of the concrete but there was a proviso that there should be no treading on the first layer of concrete while laying the reinforcement. This would have necessitated the use of a bridge and would have involved the setting back of the spreader and frequent adjustment of the spreading level of the machine. The contractor therefore decided to spread the concrete in a single layer and it was thus necessary to lay the reinforcement in place at its correct depth, namely, $2\frac{1}{2}$ in. from the top surface, ahead of the spreading machine. The use of wire supporting stirrups during the concreting of the trial slab did not prove satisfactory, the fabric sagging between supports. This was confirmed when cores were cut, the steel varying in depth from 3 in. to 6 in. In addition, an undesirable delay was caused by the placing of the mats.

The contractor then introduced a "sledge" arrangement to support the reinforcement. This is shown in Fig. 4 and was simply a wooden framework having runners about 12 ft long parallel to the length of the road and cross-members to support the reinforcement, the whole being pushed forward beneath the reinforcement by the spreader. The depth of the sledge was such that the reinforcing mats were supported at a level slightly higher than the design position to compensate for their being depressed as the weight of the spread concrete was applied and the sledge support withdrawn. The gap between the leading edge of the spread concrete and the back support of the sledge was about 2 ft. With the paver running on the formation it was not possible to place more than one set of three mats at a time. This was no more than had been possible with stirrups and it was still necessary for the spreader to stop while the next set of mats was placed, but the time taken was less. When, however, at a latter stage in the work the paver was run outside the forms it was possible to lay the mats out, tied together at the overlaps, for almost any distance ahead of concreting operations. The reinforcement then ceased to be a cause of stoppages to the progress of the spreading machine. At the end of each slab the sledge was pulled forward clear of the expansion joint and the last few feet of reinforcement supported on stirrups. Upon starting or restarting it was necessary to fix the reinforcement temporarily with pins driven into the base to prevent it from being pushed forward during spreading of the first few feet of concrete.

Spreading

To the Author's knowledge, concreting to a full width of 24 ft has not previously been attempted in Britain and it required the adaptation of existing plant. As already mentioned, the contractor used spreading equipment which was in process of development with a view to attaining uniformity of spread over the whole area. A pair of right- and left-hand helical screw blades, independently controlled, spread the concrete to either side from a hopper built over the screws at the middle of the span and fed directly from the paver mixer. Generally speaking this equipment proved satisfactory, but had certain drawbacks which could be overcome in a fully developed model. These were:—

(*a*) The main drive was not strong enough to allow full-sized batches of concrete to be fed into the spreader without risk of failure of the drive. Batches of 1 cu. yd could be and were used when spreading operations were proceeding without any interruption, but very often each batch was delivered to the spreader in two portions.

(*b*) When for some reason or other the spreading operation had to be stopped while the hopper was filled with concrete, the restarting of the spreading screws in the stiffening concrete sometimes imposed a load which caused failure of the main drive.

A strike-off screed was incorporated in the spreading device and this was adjustable for height. Throughout the scheme a surcharge of 2 in. on the high side and 1½ in. on the low side of the carriageway was found to be necessary.

Finishing

The finisher used was of a type whose operating width can be varied between 8 and 27 ft. It has an oscillating strike-off screed at the front of the machine, a vibrating beam, and an oscillating smoothing beam. The oscillating strike-off and smoothing beam can be extended within the above range but for each slab width a separate vibrating beam is required.

Observation of the performance of the finishing machine indicated that the resulting profile of the concrete could be improved and the following modifications to the original machine were carried out.

(1) The sideways oscillations of the smoothing beam were reduced to about 1 in.

(2) The length of the smoothing beam was reduced to ensure that the surface of concrete was screeded from forms only and not from the independent rails on which the machine was travelled.

(3) The weight of the smoothing beam was increased so that the high spots were planed off and there was less tendency for the beam to ride over them.

(4) The width of the beam was increased at the points of contact with the formwork to enable the beam to "ride" the dips at the end of each form and so minimize the source of surface irregularity.

After the final pass of the smoothing beam the surface of the concrete was brushed transversely with a soft broom to leave light brush marks with a view to providing suitable resistance to skidding. Tests on the coefficient of friction are described later (Table 6).

Wet-surface profilometer

A wet-surface profilometer, to record on four different tracks the surface profile of the concrete directly after the passage of the finishing machine, was provided and

operated by the Road Research Laboratory and proved to be invaluable in indicating irregularities in the finished surface, however caused (Fig. 3). Where the records of the wet-surface profilometer indicated poor profiles additional passes were made with the oscillating finishing beam. Generally, however, the profile was governed by the first pass. Some improvement was effected by a second pass but subsequent passes produced negligible change. It is considered that a simpler version of this apparatus would prove to be a valuable addition to the equipment of a finishing machine.

The fact that the contractor's foreman and finisher driver referred regularly to the profiles indicates that they were of real practical assistance on the job, and were not merely useful as technical data. The profilometer records are discussed later in the paragraph on surface irregularity measurements (see p. 218).

Formation of joints

(*a*) *Expansion joints*

The basic method of forming the expansion joints is shown in Figs 2c and 5, a rigid assembly being obtained by the mutual support of the softwood filler ($\frac{3}{4}$ in. thick), the dowel bars of 1 in. dia. (which had a large washer welded at the mid-point), and by specially made supports to which the ends of the dowel bars were wired. Two such assembles, each 12 ft long, were used at each intermediate joint. For end-of-day joints only one wire support was used, the projecting ends of the dowel bars either being supported by timbers or left unsupported.

To give greater rigidity in the assembly of the expansion joints, $\frac{3}{4}$-in. softwood filler was used in place of fibrous jointing material. During the early stages of the work, the wooden filler was surmounted by a metal capping strip which was withdrawn after the finisher had passed over. It was found, however, that immediately adjoining the joint the finished concrete level was unduly high owing to surge when the vibrating beam passed over and, also, when the capping was withdrawn, there was a tendency to accentuate the surface irregularity by lifting the concrete on either side. An attempt was then made with a wooden strip screwed to the top of the filler board —this could be lifted without disturbing the concrete, but the surge bump was still present. A third trial method was to remove the wooden strip after the vibrator had passed once and to fill the groove left with a weak sand-cement mix and then make one more pass with the oscillating beam to remove the surge bump. The cement mortar was removed the next day. This gave a reasonable result, but it was difficult to leave the wooden strip in position long enough to allow the concrete to harden sufficiently to form a clean arris, and a spalled edge resulted. Finally, it was decided that to get a true surface grade over the joint it would be necessary to concrete over the filler and then cut down to it, when the concrete had hardened. A small triangular wooden fillet was, therefore, nailed to the top of the filler board, the apex of the fillet being $\frac{3}{4}$ in. below the surface of the concrete, thus avoiding any surge bump. The pointed top of the wooden fillet induced a reasonably straight shrinkage crack in the concrete and the joint was finally finished off to 1-in. overall width with a diamond saw about one week after placing (see Fig. 2c).

Records of the Road Research Laboratory's multi-wheel profilometer showed that this latter method had been successful in substantially reducing the irregularities at the joints.

Expansion joints were sealed, after priming, with a rubber-bitumen joint-sealing compound complying with B.S.2499 : 1954.

(*b*) *Longitudinal joint*

To avoid any tendency for the slabs to crack longitudinally a centre joint $\frac{1}{4}$ in. wide and 2 in. deep was sawn with a diamond saw about 2 weeks after placing. At the outset the speed of sawing a 2-in.-deep groove with the machine used was about $\frac{1}{2}$ ft/min and the cost of diamond-saw blades worked out at about 1*s*/ft per 2-in. depth. This rate of progress was later speeded up considerably, however, and in fact the last 500 linear ft were sawn in 7 hours with a 16-in.-dia. blade. The longitudinal joint was sealed with the same compound as that used for expansion joints.

Curing

Curing of the concrete was by spraying with a proprietary resinous material at a coverage of about 25 sq. yd to the gallon to form a waterproof membrane. During the spraying the sides of the concrete slab along which the white edges were to be laid were protected by portable covers of waterproof paper fastened to light wooden frames. The curing membrane was, in general, sprayed on about 1 to 2 hours after the concrete at any point was first spread.

Fortunately, throughout the work the weather was such that no protection by way of frames, tents, or otherwise was required immediately following the finishing of the concrete. Concreting continued until the 5th October, 1955, during an almost uninterrupted spell of fine weather. So far the efficacy of the curing can be gauged only by the fact that no initial crazing or cracking of the slab surface has taken place even under conditions of hot sun and drying winds.

White edging

The design provided for definition of the edge of the carriageway with a white edging strip alongside hard flush shoulders, similar to Continental practice. The white concrete edge strips were formed upstanding $\frac{1}{2}$ in. above the concrete slab and were laid while the main concrete was still fresh to ensure monolithic construction. The operation followed that of curing and lagged behind the last finishing pass at any point by a few hours. The strip was formed in lengths of 10 ft, each end of the 10-ft length being ramped down to slab level. These ramps serve a threefold purpose; as a drainage channel on the low side of the slab, as a warning to drivers of vehicles that a wheel is riding off the carriageway proper, and as facets to give greater reflexion of headlights on the white edging (Fig. 2d).

Initially it was intended to tone down the natural cement colour of the surface of the main concrete slab by the use of either black or red iron oxide and so provide greater contrast with the white edge strips. On reconsideration, however, it was thought undesirable to reduce the value of the light cement colour under night-driving conditions, and experiments were directed towards achieving the greatest contrast between the slab and the white edge strips.

It was found that the incorporating of titanium oxide with a concrete whose other components were all essentially white (white cement and crushed calcined flint), gave an intensity of whiteness which was a sufficient contrast to the grey colour of the main slab. Five per cent by weight of titanium oxide was added to the white cement, the crushed calcined flint was graded $\frac{3}{8}$ in. to dust, and the concrete mix was one part by weight of white cement to four parts by weight of calcined flint.

A view of the finished work is shown in Fig. 6, between pp. 218 and 219.

Comments on construction methods

The selected contractor had considerable experience of the construction of airfields with heavy spreading and finishing equipment and had devoted a great deal of effort to developing methods whereby many of the observed imperfections of the system employed could be removed. The Oxton extension, however, required the development of rather different methods for the following reasons :—

(1) The space at the disposal of the contractor for the deployment of his equipment was much more restricted than on an airfield site.
(2) The slab width of 24 ft was greater than had previously been laid in one operation.
(3) The concrete slab was to be reinforced.
(4) All expansion joints were to be fully dowelled.

Comment has already been made on the steps which were necessary to overcome the limitations imposed by the reduced working space, but the fact that the slab width was 24 ft as opposed to the maximum of 20 ft in airfields did not bring any major problems in its train.

As regards point (3), the methods employed to support the reinforcement have been described. Measurements to determine the ultimate position of the reinforcement after compacting the concrete were made at a number of places, and these showed the steel placed by the sledge to be within a reasonable tolerance of the correct position.

The construction of joints in concrete slabs in such a manner as to avoid any major irregularity in the surface profile of the slab has always presented a major problem and much of the bad riding quality of concrete surfaces can be closely associated with joints. In unreinforced slabs such as are used in airfields, the use of a vibrating cutting blade forced into the unhardened concrete has, it is understood, brought about an improvement in the riding quality in the neighbourhood of joints but has not proved to be the complete answer to the problem.

Where dowelled expansion joints are to be used the difficulty of ensuring that the uniformity of the slab surfaces in the vicinity of joints is not impaired is a very real one, and it would be true to say that at Oxton problems associated with joint construction were the most difficult to solve.

Inspection of the profiles of the finished surface will show how great a problem the construction of dowelled joints—both running joints and those at the end of a day's work—presented. Towards the end of the work, however, it can be said that the problem of construction of running joints was improved to such an extent that it is virtually impossible to determine their position in the profile. The riding quality in the neighbourhood of the joint between two successive days work, however, is generally not so good and these joints can fairly readily be detected.

The irregularities in the profile which can be associated with running joints might be placed in two categories:—

(*a*) Those caused by the halting of the forward progress of the concreting equipment while all the work associated with the proper assembly of the joint in its correct position is completed.
(*b*) Those associated with the joint itself, e.g., the obstruction offered to the forward movement of concrete under the action of the vibrating and smoothing beams by the presence of the joint filler.

As regards (*a*) if for any reason the concreting operations were delayed then irregularities in the surface profile were made manifest. In extreme cases the correct positioning of a dowel-bar joint assembly occasioned a delay in operations of ½ hour, during which time the spread but uncompacted concrete would be hardening and becoming less easy to compact and finish. As mentioned earlier, the presence of concrete in the hopper of the spreader while spreading operations were held up sometimes resulted in the breakage of the main-drive chain. The repair of this occasioned still further delays.

It is suggested that this could be overcome by truing up the formation to an accuracy of ⅛ in. at selected points by means of a thin bed of mortar laid on the formation, so that when the time comes to place a joint assembly this can be done with the minimum of delay. Since it is not possible to position the reinforcement so accurately that the end of a mat will precisely coincide with the position of a joint, then provision should be made for cutting short the reinforcement or inserting short lengths where necessary; alternatively the bed of mortar might be sufficiently wide to allow some latitude for positioning the joint in relation to the actual end of the reinforcement mats. Then, as a complement to that, the method finally adopted to give a completely uniform profile would be used, namely, covering the joint filler with a triangular-shaped wooden fillet, passing the finishing machine straight over the assembly, and, when the concrete had hardened, cutting the joint by mechanical saw.

Field tests

Tests were undertaken on the site to determine the compaction factor of the concrete and the moisture content of the sand. These are described on p. 209.

Tests on concrete

(*a*) *Test beams*

From the commencement of concrete-laying four 6-in. × 6-in. × 28-in. beams were made each day, two in the morning and two in the afternoon, each pair being made from a single batch of concrete. The beams were cured under water and tested for flexural strength in accordance with B.S.1881 at 28 days. The results of these tests are set out in Table 2, the figures being read in relation to the requirements under the specification, namely, average compressive strength of each group of two beams not less than 5,000 lb/sq. in. and average flexural strength of each pair of beams not less than 600 lb/sq. in.

(*b*) *Test cores*

Test cores were cut, six from the trial slab and seven from the main slab, in selected locations and the minimum strength of cores in accordance with B.S.1881, 1952 was specified as equal to a minimum equivalent-cube strength of 5,000 lb/sq. in. at 28 days. Results of tests of cores from the main slab are set out in Table 3.

These cores were taken generally from slabs where the test beams gave poor results. With the exception of a few small or medium water voids the compaction was very good in all cases (Fig. 7).

The average approximate 28-day-cube stress for all the cores was 7,050 lb/sq. in., and with regard to core No. 3 it is considered that the comparatively low figure of 4,200 lb/sq. in. was due to the presence of a piece of reinforcement, quite near to the periphery of the core, which may have reduced the effective section at the point of failure.

TABLE 3.—TESTS ON CORES

Core mark	Position (chainage): ft	Date concrete laid 1955	Date core tested 1955	Height: in.		Depth of reinforcement: in.	Crushing strength: lb/sq. in.	
				Max.	Min.		Estimated equivalent	Approximate 28-day-cube stress
OE1	2,665	15th Sept.	15th Nov.	8·7	8·1	3	5,950	5,200
OE2	2,960	16th ,,	,,	8·35	7·9	$2\frac{3}{4}$	10,000	8,900
OE3	3,270	19th ,,	,,	8·7	8·1	$2\frac{1}{2}$	4,800	4,200
OE4	3,490	22nd ,,	,,	7·8	7·3	$2\frac{1}{4}$	7,100	6,300
OE5	3,860	23rd ,,	,,	7·7	7·7	$3\frac{3}{8}$	8,750	7,800
OE6	4,795	29th ,,	,,	9·1	7·7	$4\frac{3}{8}$	8,150	7,800
OE7	5,425	4th Oct.	,,	9·4	8·55	$5\frac{3}{8}$	10,300	9,800

SURFACE IRREGULARITY MEASUREMENTS

Profiles of the road surface were obtained by the Road Research Laboratory using:

(*a*) a wet-surface profilometer towed behind the compacting machine;
and (*b*) the multi-wheeled profilometer on the hardened concrete.

The results from the wet-surface profilometer gave an immediate indication of the irregularity of the surface being produced and led to adjustments in technique as the job progressed. It recorded the profiles at four positions across the width of the slab, namely, at 3 ft and 9 ft from each edge; these positions would be approximately those of the tracks of vehicles using the road. The wet-surface profilometer ran on the rails which formed its datum and the records from this apparatus did not, therefore, include any irregularities which may be attributable to errors in the shape and alignment of the forms or to movements of the forms during the laying and compacting of the concrete. The multi-wheeled profilometer gives an overall measurement of the irregularity of the surface in terms of the value of an irregularity index q and in relating the values obtained to riding-surface quality as experienced by average drivers, it has been indicated by tests carried out by the Road Research Laboratory that a q value of less than 75 in/mile represents "good or better" riding quality, of 75 to 130 in/mile "fairly good to very fair," 130 to 200 in/mile "indifferent to rather poor," and over 200 in/mile "poor to bad (deserves remedial treatment)." Summarized results of the irregularity index values are given in Table 4.

It will be noticed that whilst, in general, the values indicate a riding surface which can be called "good or better," it is not as good as it was hoped to achieve. It is to be noted, however, that improvement was obtained as the job progressed and that whereas during the first section q values of 70 in. to 80 in/mile were obtained, these were fairly rapidly reduced to 50 in. to 60 in/mile. Since the concreting was carried out over a period of only 17 days it is reasonable to suppose that better riding qualities would have been achieved with more experience; also that contractors experienced in pavement work should be able to produce q values of less than 40 in/mile once they have had some experience in the laying of concrete roads as distinct from airfields.

It may also be noted that the q values are rather lower near the edges than in the

FIG. 3—GENERAL LAYOUT OF PLANT

FIG. 4.—VIEW OF SPREADER, SHOWING METHOD OF SUPPORTING REINFORCEMENT

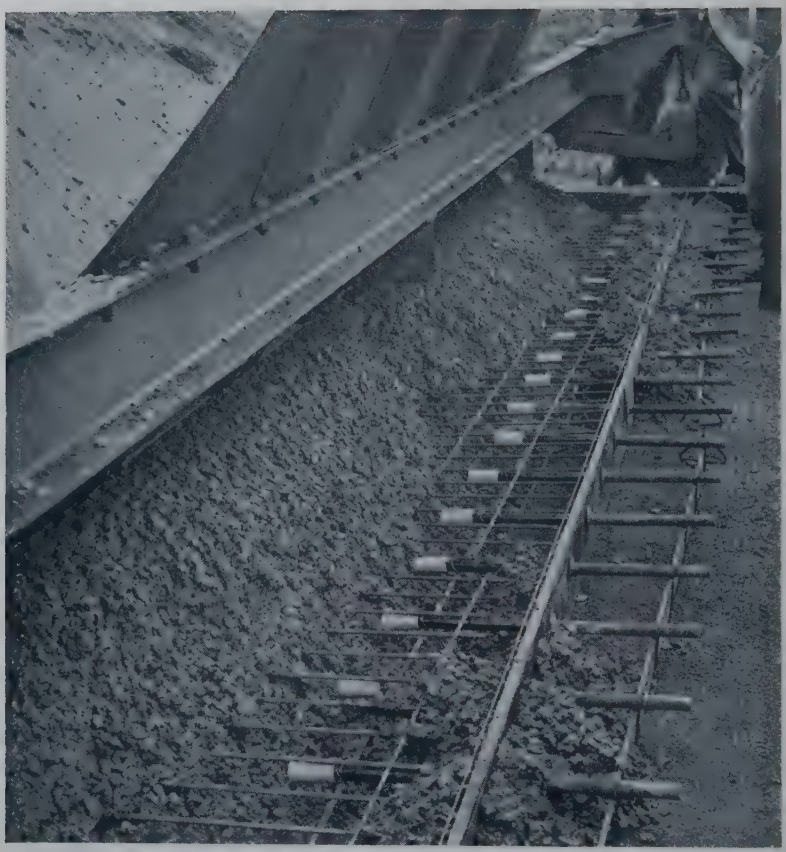


FIG. 5.—DOWEL-BAR ASSEMBLY AT EXPANSION JOINTS

FIG. 6.—GENERAL VIEW OF FINISHED WORK

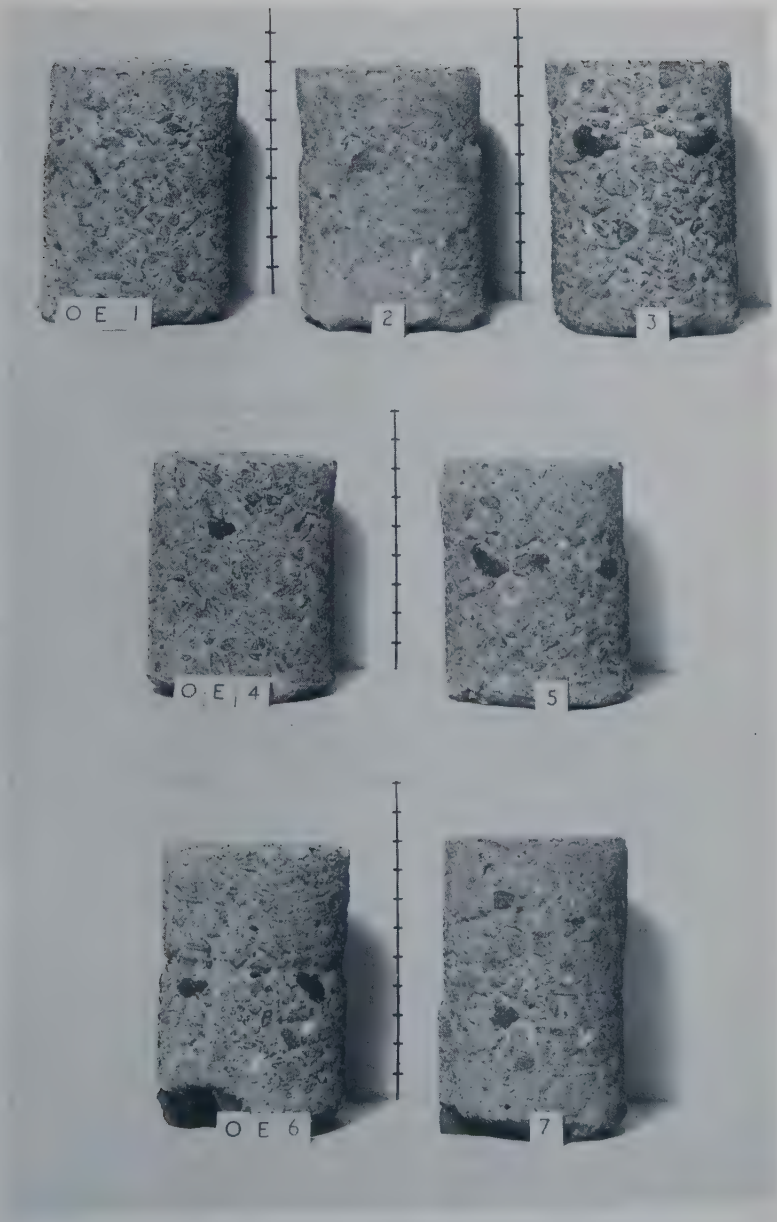


FIG. 7.—6-IN. TEST CORES TAKEN FROM MAIN SLAB

TABLE 4.—RESULTS OF SURFACE IRREGULARITY MEASUREMENTS BY MULTI-WHEELED PROFILOMETER

Slab Nos	Section No.	Index of irregularity q: in/mile			
		3 ft from west edge	9 ft from west edge	9 ft from east edge	3 ft from east edge
1–5	1	70	78	72	67
6–9	2	47	73	63	55
11–15	3	45	63	58	53
16–20	4	48	62	58	53
21–24	5	59	64	59	45
23–26	—	54	49	49	40
25–29	6	49	55	56	44
30–34	7	73	81	78	77

Note:—The readings are corrected for unfilled joints but not for total joint effect.
Values of q for slabs Nos 23–26 were evaluated from the profiles; this length was considered to be the best day's work as judged by wet-surface profilometer records.

central region. The reason for this is not known but may perhaps be associated with the stiffness of the vibrating beam and the oscillating screed or with the degree of uniformity of the spreading.

In slabs 23, 24, and 25 the wet-surface profilometer indicated very little irregularity indeed over the whole day's run. The q values for this length were, therefore, determined separately from the multi-wheeled profilometer records and showed values of 40 in. to 50 in/mile. It seems likely that a large part of this irregularity was caused by rail and form setting.

The records on the hardened concrete were also examined to determine the size of the irregularity at the joints at the same four positions as were recorded by the wet-surface profilometer. These results are given in Table 5. For the stop-end joints (i.e., those at the end of a day's work) and for those joints constructed during the day these can be assessed in accordance with the key given at the foot of Table 5. Table 5a shows the percentage of joints falling under certain categories for the different forms of construction. It will be noted that almost without exception the stop-end joints can be considered "poor" or "bad." The joints formed during the day's work in the early part of the job were predominantly "poor" to "fair" whilst those in which the top of the slab was cut after the concrete had hardened (i.e., from joint 14/15 onwards) were "fair" to "good." The greater irregularity at the stop-end joints is attributed to the fact that the concrete tended to run away from the vibrator as it was approaching the stop-end leaving a shortage of concrete for compaction. Although after the first pass of the machine further concrete was added, it seemed that the technique had not been developed to the point where good riding quality could be obtained at the end of the day's run of concreting. The formation of stop-end joints and, to a lesser extent, of expansion joints during the day's run is clearly a matter which would repay further investigation.

Profiles over some of the joints are given in Fig. 8a. It will be noted that those at stop-ends may be very bad (joints 20/19 and 32/31) and that even those of average merit are not very good (joints 19/18 and 28/27). The best joint profiles along the road were obtained over joints in continuous construction (joints 18/17 and 25/24)

TABLE 5.—SIZES OF IRREGULARITIES (INCHES) IN THE IMMEDIATE VICINITY OF JOINTS

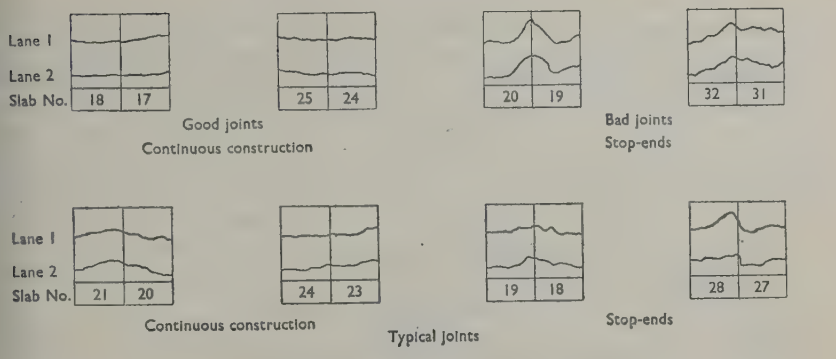
Joint	Stop-end joints				Continuous construction joints			
	Lane 4	Lane 3	Lane 2	Lane 1	Lane 4	Lane 3	Lane 2	Lane 1
1/2					+ 0·08	+ 0·15	+ 0·20	+ 0·35
2/3	− 0·15	+ 0·15	+ 0·30	+ 0·35				
3/4	+ 0·15	− 0·23	+ 0·20	+ 0·30				
4/5					− 0·05	− 0·10	+ 0·10	− 0·15
5/6	+ 0·25	+ 0·30	+ 0·25	+ 0·20				
6/7					− 0·10	+ 0·15	+ 0·20	+ 0·10
7/8	+ 0·10	+ 0·15	+ 0·30	+ 0·20				
8/9					+ 0·25	+ 0·10	+ 0·30	+ 0·25
9/10								
10/11								
11/12					− 0·13	+ 0·10	+ 0·15	+ 0·07
12/13	+ 0·20	+ 0·20	+ 0·20	+ 0·13				
13/14	+ 0·15	− 0·15	+ 0·12	+ 0·06				
14/15					+ 0·03	—	− 0·05	− 0·05
15/16	+ 0·35	+ 0·25	+ 0·20	+ 0·20				
16/17					− 0·10	− 0·05	− 0·10	—
17/18					+ 0·15	—	—	− 0·10
18/19	+ 0·20	+ 0·23	+ 0·20	+ 0·15				
19/20	+ 0·25	+ 0·25	+ 0·30	+ 0·35				
20/21					+ 0·10	+ 0·18	+ 0·15	+ 0·15
21/22					− 0·06	− 0·10	+ 0·10	− 0·05
22/23	+ 0·10	− 0·20	− 0·30	− 0·25				
23/24					− 0·07	− 0·03	− 0·05	− 0·05
24/25					—	− 0·04	− 0·03	—
25/26	+ 0·15	+ 0·22	− 0·05	− 0·10				
26/27					− 0·05	—	+ 0·08	—
27/28	+ 0·20	+ 0·20	+ 0·15	+ 0·25				
28/29					− 0·10	+ 0·15	+ 0·20	+ 0·15
29/30	+ 0·25	− 0·10	− 0·15	− 0·15				
30/31					+ 0·03	+ 0·05	+ 0·05	+ 0·10
31/32	+ 0·20	+ 0·20	+ 0·20	+ 0·35				
32/33	0·0	0·0						
33/34	− 0·10	+ 0·20	+ 0·15	+ 0·20				

Note:—The sizes of the irregularities in the immediate vicinity of the joints are shown above. These irregularities may or may not be entirely due to the effect of the joints. It is not possible in the above Table to indicate whether the irregularities are of long or short pitch, or the nature of their shape.

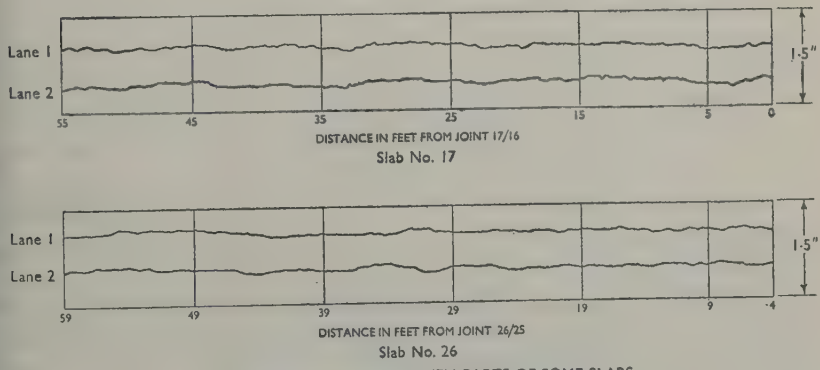
As an indication of the order of riding quality, irregularities of 0·00 in. to 0·05 in. are considered to be good, 0·06 in. to 0·14 in. fair, 0·15 in. to 0·20 in. poor, and greater than 0·20 in. bad.

and in no case with such joints was the profile as bad as those for the stop-ends shown in joints 20/19 and 32/31. It seems that when constructing expansion joints during the day's run there need be no inherent difficulty in obtaining good profiles. So far as stop-ends are concerned, however, an improvement is required in the technique.

Profiles of lengths lying between joints are shown in Figs 8b and 9a. In Fig. 8b



(a) IRREGULARITIES IN THE VICINITY OF JOINTS

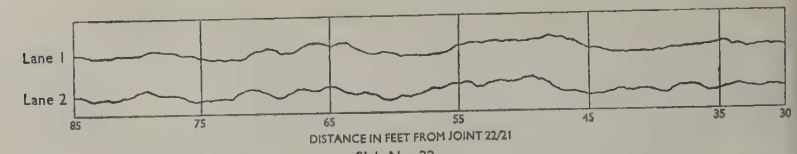


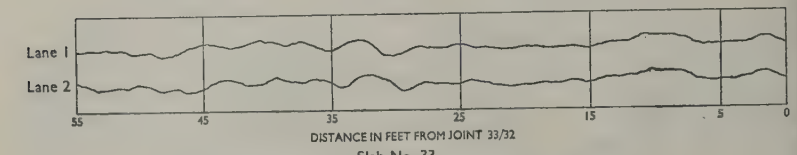
(b) PROFILES OF THE MORE EVEN PARTS OF SOME SLABS

FIG. 8.—PROFILES AT JOINTS AND PROFILOMETER RUNS

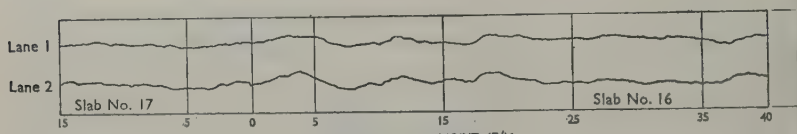
TABLE 5a.—SUMMARY OF IRREGULARITIES AT JOINTS

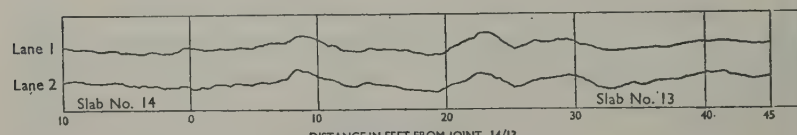
Type of joint	Percentage classified as			
	bad	poor	fair	good
Formed joint at end of day's work .	37	55	8	—
Formed joint in the middle of day's work	20	50	30	—
Cut joint in the middle of a day's work	—	11	50	39





(a) PROFILES OF THE MORE UNEVEN PARTS OF SOME SLABS





(b) PROFILES OF PARTS OF SLABS NEAR JOINTS

FIG. 9.—PROFILES AT JOINTS AND PROFILOMETER RUNS

are profiles typical of the best surfaces produced and in Fig. 9a those of the less satisfactory surfaces. In Fig. 9b are shown the profiles near two joints selected at random, one being in continuous construction (17/16) and one near a stop-end (14/13). In both cases there is evidence of waviness for about 20 to 30 ft back from the joint and this is particularly marked in the case of the stop-end. It may be significant that the spreading and finishing machines were stopped for some time within this region while the joint was being prepared.

As an exercise, the profiles of the best lengths of road between joints were selected for analysis and this length amounted to 584 ft, i.e., about 15% of the total length of the road. The q values for the four positions were 23, 35, 31, and 32 in/mile respectively. Whilst it is not submitted that these values represent in any way the riding-surface quality of the road as a whole, they do suggest that a maximum q value of 40 in/mile was not an unreasonable target to specify and that such a target should be attained after more experience in constructing roads of this type.

Tests on coefficient of friction

Tests for resistance to skidding were carried out by the Road Research Laboratory on three sections of the road soon after the concreting and joint sealing were completed, but before the road was opened to traffic. These tests were taken on clean surfaces which had been thoroughly wetted. The results are indicated in Table 6.

The good results obtained are thought to be due to the brushed surface finish and to the angular granite aggregate used for the concrete. This is borne out by tests taken during 1948 on the experimental road, where a rounded gravel aggregate was used. The sideway force coefficient at 30 m.p.h. in this case was 0·49 on the brushed surface and 0·34 on the surface as left by the finishing beam.

Table 6.—Resistance to skidding

Section No.	Sideway force coefficient						Range at 30 m.p.h.	Skidding distance from 30 m.p.h.: ft
	Speed: m.p.h.							
	30	25	20	15	10	5		
1 (north-east end)	0·62	0·69	0·76	0·82	0·86	0·87	0·57 to 0·69	41
2 (centre)	0·66	0·74	0·81	0·87	0·89	0·89	0·61 to 0·77	38
3 (south-west end)	0·73	0·79	0·80	0·84	0·86	0·88	0·71 to 0·75	37

Costs

It has already been indicated that in this work economic considerations were sacrificed in the interest of gaining experience of the application of recent trends in design and the use of machinery which has formerly been engaged in Britain only on large-scale construction of airfield runways. It is inappropriate, therefore, on the basis of the cost of the work in question, to make comparisons with the cost of other forms of construction or in fact to draw any conclusions as to the probable cost of large-scale construction to a specification applied to this relatively short work. The economic test could be applied only as and when any large-scale concrete-road construction is undertaken and even then a test of initial cost only would be inappropriate. The test must be initial cost plus maintenance costs for a period of at least 40 years.

Conclusions

The following are the main conclusions arrived at from the construction of this road:—

(1) The contractor after a very short time was able to work to a surface irregularity value (q) of well under 50 in/mile. A target of 40 in/mile is therefore reasonable and should be achieved once the contractor has overcome his initial difficulties and settled down to regular routine construction.

(2) A device such as the wet-surface profilometer would be of great assistance to engineers and operatives in pin-pointing areas where some attention is required to correct irregularities while the concrete is still plastic. Machinery manufacturers could well assist in this matter.

(3) The new type of distributor developed by the contractors shows considerable promise in providing a concrete which is uniformly spread.

(4) Very good compaction of the concrete is possible throughout the full depth of the slab when concrete is spread in a single layer up to 8 in. thickness.

(5) Manufacturers of forms should give attention to the importance of straightness, so that warped new forms are not sent out. A maximum tolerance of $\frac{1}{16}$ in. in 10 ft should be achieved.

(6) A method of forming expansion joints during the day's work was evolved which had little, if any, effect in producing irregularities.

(7) There is need to develop a method of forming joints at stop ends so as to avoid the formation of surface irregularities.

(8) The placing of reinforcement has been shown to be amenable to mechanical manipulation and need present no problem in future work.

(9) The importance of maintaining a smooth forward progress of plant, with as few stoppages or delays as possible, has been demonstrated. This implies that all construction plant should operate from outside the forms.

Acknowledgements

The Author acknowledges the assistance and advice given in the design of these works and in the preparation of this Paper by the Director of Road Research and several officers of the Road Research Laboratory and the Ministry of Transport, to whom he is greatly indebted. He must also acknowledge with grateful thanks the very valuable co-operation of the Contractors, Messrs John Laing & Son Ltd, and their individual directors and representatives. All of them have made a valuable contribution to the experience gained.

Tables 3, 4, 5, 5a, and Figs 7, 8, and 9 are reproduced by permission of the Director of Road Research. The photographs Figs 3, 4, 5, and 6 are included with the permission of Messrs John Laing & Son Ltd.

The Paper, which was received on 2 December, 1955, is accompanied by five photographs and three sheets of drawings, from which the half-tone page plates and the Figures in the text have been prepared.

Discussion

The Author introduced the Paper with the aid of a film.

Dr W. H. Glanville (Director of Road Research, D.S.I.R.) observed that the essence of concrete-road construction was care and attention to detail, and the Author's experiment in Nottinghamshire was a good example of the great care that was necessary. Despite the machinery employed, it remained a skilled job.

The first Barber-Greene finishing machines for bituminous roads introduced into the United Kingdom had produced results which were not always up to the standard of roads made by hand. With the experience that had now been obtained with Barber-Greene machines over many hundreds and perhaps thousands of miles of roads, a first-class result could be produced which, as the Author had indicated, really set the standard for a concrete road. The concrete road, on the other hand, had not had the same chance—a few miles had been constructed here and there, but the $\frac{3}{4}$ mile described in the Paper was quite a fair-sized job compared with what had been done.

The Author, however, had shown quite definitely that excellent results could be obtained. Given care and experience, the running surface obtained could equal the best roads in

Britain. Results of 25 in/mile and 30 in/mile were shown as the best examples. There was no reason why those figures should not be obtained consistently when there were construction jobs of bigger size and longer lengths to be undertaken.

On the question of formwork, Dr Glanville agreed with the Author that the tolerance of $\frac{1}{8}$ in. was not good enough. There did not seem to be any reason why the forms should ever be a cause of irregularity. He would have thought that the irregularities could well have been as little as the $\frac{1}{16}$ in. which the Author mentioned. Again, it should be possible to set the forms accurately. Therefore, it should be feasible to get rid of the irregularities that arose from the formwork.

With reference to joint construction, Dr Glanville thought the Author had gone farther along the road to solving the problem than he had said, because his method of inducing a crack over the jointing material by the use of a pointed wedge of wood and then cutting it with the machine seemed to have been highly successful once the technique was determined, so successful that in the records the position of the joints in the best cases could hardly be detected. Even if the problem of joint construction was not yet completely solved, it could at least be regarded as possible to eliminate irregularities arising from joints.

One of the most difficult problems was the question of continuity of construction—continuity of uniformity. Laying bituminous roads was a far easier task than concrete; a little more could be added where required and rolled in, but that could not be done with concrete. Concreting had to be a continuous smoothly working process and if it was not so organized the hold-ups would cause the concrete to harden and stiffen, and once that started it was difficult to get rid of the bumps which occurred. The question of continuity needed attention. No doubt on a bigger job it would be possible to organize the work so that the continuity was better maintained.

There were examples of roads in Denmark and other countries where the irregularities were very small indeed, and the riding qualities of those roads were quite as good as the Barber-Greene roads in Britain.

When Dr Glanville had visited Sweden some years ago road constructors there had told him that their way of reducing the number of bumps was to give the men a big bonus and then subtract a given proportion of it for every bump discovered; it was found that the men rapidly acquired a technique for avoiding bumps.

Mr J. M. Fisher (Technical Adviser, John Laing & Son, Ltd, civil engineering contractors, London) remarked that the subject of the Paper was likely in the near future to be of great interest to highway engineers. Presumably the roads which they would be designing and constructing would follow the same general pattern as the road at Oxton, and, therefore, the lessons learned at Oxton would be immediately applicable. It was only a small job, the amount involved being well under £50,000, but the lessons learned from the construction of the road at Oxton might well be applicable to lengths of road many times greater.

Tenders for the construction of the road had been invited from selected contractors with a great deal of experience of airfield construction. One valuable consequence of that procedure was that it had been possible for the client and the contractor to discuss the specification in some detail, so that general agreement had been obtained on the best techniques to employ and how they could best be applied. There was no doubt that the method of having a selected list of contractors was likely to result in better work in the future.

With regard to the forms used, Mr Fisher said most of them were not new but had recently been modified and reconditioned. The major fault in them, a dip between the end of the form and the first stake housing, had apparently been caused during previous use owing to their having a sliding locking plate which had been an imperfect fit; that had led to some deflexion at those points and some distortion had been induced by the heavy loading. Unfortunately, it had not proved possible to eliminate the distortion from the screeding edge of the forms when they were sent for reconditioning, but the incorporation

of a new locking device had prevented any accentuation of the fault during the progress of the work. An inspection of new forms had recently been carried out at the manufacturer's works, and examination of a number of them had shown that the maximum departure from a true straight-edge was a good deal less than $\frac{1}{16}$ in. for a 10-ft length. Therefore, it would seem that new forms when delivered from the manufacturer would, in future, comply with one of the recommendations in the conclusions in the Paper. It was important that every precaution should be taken on the site to ensure that distortion in the forms did not occur because of mishandling there.

It was mentioned in the Paper that a great deal of attention was paid to the levelling of the individual forms and that, with the material available, a high standard of accuracy in form laying was obtained. He felt, however, that an inordinate amount of engineer's time was occupied in setting the forms, and he wondered whether somebody could suggest a device, as easy to use as boning rods, which would give as much precision in setting forms as would a precise levelling.

With regard to the spreading of the concrete and the laying of the reinforcement, the rather novel device used for the positioning and supporting of the reinforcement while the concrete was spread around it was found to be very effective, but it could only be used with the particular type of spreader employed at Oxton where the advancing face of concrete was transverse to the line of the road. Attention should be paid to that point, because in conclusion (8) on p. 224 the Author implied that all the problems relating to the mechanized handling of reinforcement were virtually solved. It was true to say that there were other means of supporting reinforcement while the concrete was spread around it, but they did not appear to be as simple as the method used at Oxton.

With regard to the results obtained with the spreader, there was little factual evidence that it attained a uniform degree of spreading throughout the whole width of the road, but there was a good deal of circumstantial evidence that it did. The circumstantial evidence was that there was a high degree of accuracy in the surface profile of the road towards the end of it. Given that the form laying was reasonably accurate, the accuracy of spreading made a very great contribution to the attainment of a regular profile.

Another feature of that type of spreader was that the movement of the concrete outwards from the centre of the spreader box to the outside of the road tended to feed it under the reinforcement, and the evidence from the cores was that uniform spreading of the concrete throughout the whole width of the road was achieved. There was good compaction throughout the depth and all round the reinforcement.

He wished, in passing, to mention the wet-surface profilometer supplied by the Road Research Laboratory and operated by its staff. The machine was of very great merit. It was noticed during the construction that the men tended to refer to it whenever they were doing passes with the machine, and it developed in them a great sense of pride in improving on their previous work. He heartily endorsed the Author's recommendation that manufacturers should give some thought to fitting machines with simple versions of the device.

The Author had dealt at length with the construction of joints and their effect on the profile of the road, and quite rightly so, because most of the concrete roads in Britain were characterized by the frequency and bumpiness of joints. Was not the poor reputation of concrete roads largely founded on those two factors?

There was much discussion at the present time regarding expansion-joint spacing, and the general opinion in Britain seemed to be that the joints should be put in at fairly frequent intervals; but engineers in some other countries considered that they should be omitted altogether and that only dummy joints should be used. No doubt the only way to tell which was the right procedure to adopt was to let the joints stand the test of time and so ascertain whether or not they were required. However, the problem of making high-quality running expansion-joints whose presence could not be detected in the profile of the road seemed virtually to have been resolved, so that the spacing of the joints was not particularly important. What was important was the fact that, despite the building of sixteen of the stop-end joints at Oxton, no satisfactory way of making them had yet

been devised. That type of joint had to occur in every concrete road at least once a day, unless a three-shift system was adopted. Therefore, there seemed to be a double incentive to improve them. First of all, it was necessary to perfect them to the greatest extent possible with the machines at present available. Secondly, it was necessary to do as great a length of concreting in a day as was possible. That was not only a function of the rate at which concrete could be produced, but was probably a function of the overall efficiency of the plant and accuracy in all details of construction so that steady forward progress of the whole train of machines could be maintained.

The list of nine conclusions drawn up by the Author as a result of his observations on the work, with the exception of (8), to which reference had already been made, properly reflected the conclusions of the contractors. With particular reference to conclusion (1), might it not be possible to specify that a q value of, say, 50 in/mile should be obtained and that adjustments in the form of a bonus or penalty for results better or worse than that figure should be paid or deducted at the end of the works. Such a scheme would provide an incentive to improve the quality of work and would probably be welcomed by many contractors.

Mr W. F. Adams (a Senior Civil Engineer, Ministry of Transport and Civil Aviation) said he found it a little difficult to reconcile the statement in the synopsis that the strength of the concrete as specified was achieved in 82% of the specimens with the data in Table 2, if that was the information to which reference was made, for it showed that out of the thirty-one tests 26% failed to satisfy the compressive-strength criterion of 5,000 lb/sq. in. and 19% failed to satisfy the flexural-strength criterion of 600 lb/sq. in. However, that was not of practical importance, for he believed that a considerably lower figure than the 5,000 lb/sq. in. specified would have produced a perfectly satisfactory result. That high figure was specified for reasons connected with the experimental nature of the work.

It was also a little puzzling to see from Table 2 that the standard deviation in compressive strength was 847 lb/sq. in., which represented according to Stanton and Walker a good, though not excellent, standard of control, and then to look at the average strength of approximately 5,550 lb/sq. in., which, with the standard deviation, would be expected to result in minimum strengths down to 3,800 lb/sq. in., which was, in fact, realized in one case.

Mr Adams wondered whether sights had been raised high enough in designing the average strength of the mix. In fact, if 5,550 lb/sq. in. was the mean strength aimed at, it would have needed a coefficient of variation of about 5%, representing really well-controlled laboratory tests, to make sure that only one in forty fell below the 5,000-lb/sq. in. mark.

On p. 218, referring to Table 4, the Author said: "It may also be noted that the q values are rather lower near the edges than in the central region." Statistical analysis showed that the q values were very significantly lower nearer the edges. There was less than a 1 in 1,000 chance that that was a false conclusion.

In the same Table there was a highly significant variation along the length of the road for all the lanes concerned—another interesting point having a definite bearing on the prospects for the future.

Table 5 disclosed that there was a highly significant difference between the stop-end joints and the continuous joints—an obvious conclusion, but it was well to have it proved to have a significance of over 1 in 1,000.

It was also clear from Table 5 that the results obtained for the irregularity of the joints varied highly significantly along the length of the road in the case of continuous construction joints and did not vary significantly in the case of stop-end joints, which confirmed the Author's statement that continuous construction joints showed improvement as experience was gained, whilst the stop-end joints did not. That seemed to point to the need for a drastically new form of attack on the stop-end joint problem.

It was stated in conclusion (8) that "The placing of reinforcement has been shown to

be amenable to mechanical manipulation and need present no problem in future work." Mechanically speaking, that was true. However, with reference to Table 3, the number of cores in which the sledging type of placing had been used was not known, but if the depth of reinforcement and the fact that the reinforcement was placed with relation to the bottom of the slab and not the top of it were borne in mind, it remained true that there was a variation of about 2½ in. in the position of the reinforcement in the slab as shown by the cores. That appeared to point to a need for improvement.

Dr R. H. H. Kirkham (Principal Scientific Officer, Road Research Laboratory), referring to the riding quality which had been achieved at Oxton, observed that the irregularity index was the lowest for any considerable length of machine-laid road in the United Kingdom and was of the same order as that of a hand-laid concrete road nearby. Although the riding quality was "good or better" the Author had said that it was not as good as had been expected, and an examination of the profiles taken during the job might be expected to suggest some reasons for some of the remaining irregularities.

Several speakers had mentioned forms. An examination of the profiles showed that there was a regular rise and fall in most of the work of about $\frac{1}{16}$-in. at the end of each 10-ft length; that was almost certainly attributable to the lack of proper shape in the surface of the form. Although the irregularity was small, American experience suggested that a rhythmic roughness, even though of small amount, was one of the most objectionable features of irregularity for a high-speed road, and the comfort of the traveller might best be served by improving the accuracy of the form as manufactured.

A second interesting point about the profile was that there were some good slabs in all sections and some poorer slabs. An examination of the poorer slabs seemed most likely to lead to guidance as to what should be avoided in the future. One of the first things apparent was that a certain amount of uneven filling of the spreading machine had led to irregularities. Once or twice patches were left unfilled by the spreading machine and those later had to be filled by hand; in those places a bump was always left after the first pass of the finishing machine.

There was greater irregularity at the centre of the slab than at the edges although the shape of the profile was similar. The spreading machine which was used was a good deal better than any previously used in Britain but, unfortunately, it was loaded at the middle. Any uneven filling of the machine would therefore have a greater effect on the pre-compaction of the concrete at the middle of the slab than at the edges, and the increased irregularities at the centre probably arose from the type of spreading machine.

Turning to stoppages, Dr Kirkham said that once or twice, for various reasons, the spreading machine had been stopped for periods of ½ to 1 hour and some concrete which had been spread was therefore left uncompacted. In those regions there was always a bump and repeated finishing attempts did not manage to remove the whole of it. Other bumps could also be attributed to stopping and starting the finishing machine.

Changes in workability of the concrete seemed also to have affected the profile. Generally, when the concrete became more workable there was a slight rise in the surface above form level, which made finishing to form level more difficult than elsewhere.

Dr Kirkham thought the use of the wet-surface profilometer led rapidly to a reduction in surface irregularities and he was pleased to find that both the Author and Mr Fisher had found it of value. The Road Research Laboratory were at present considering further development of the recording instrument so that it could be fitted to any finishing machine, in the hope that it would prove of even more value.

The irregularities of joints were also of interest. Although the Author had classified some of them as bad in relation to the other joints at Oxton, it was not fair to say that any of the joints at Oxton was a bad joint. If they considered the average step at a joint after the first passage of the compacting machine as a measure of the irregularity—a very useful measure, since it was not affected by hand finishing—the joints at the end of the day's work at Oxton gave an average step of only a little more than 0·2 in., whilst joints formed during the day's work gave an average of 0·19 in. On an experimental

road some years ago the figures for stop-ends and joints formed during the day's work were almost exactly the same.

Both at Oxton and on the experimental road there was a certain amount of hand compaction around the dowel bars near the joint, but on another experimental road, where there was no pre-compaction around the dowel bars, the step at the stop-end joint was as high as 0·4 in. and that on a joint during the day's work as high as 0·3 in Contrary to expectation, therefore, a certain amount of pre-compaction of the concrete near the joint seemed to have led to less irregularity.

The difference measured after the first passage of the finishing machine between stop-ends and joints formed during a day's work was much less than the figure measured on the hardened surface. Dr Kirkham suspected that that was probably because of the difficulty in estimating the line of the next slab when hand finishing at the stop-end.

Could the stop-end joints have been improved, at least to the level of the other joints if a batten had been placed across the road at form level 2 ft clear of the joint so that the men forming the arris had some guidance about the level of the next slab?

The most important conclusion, on joints, was that cutting joints reduced the irregularity after the first pass to about half that with the formed joint, and that amount of irregularity was small enough for most of it to be removed during the finishing operation.

Dr A. R. Collins (Technical Director of the Cement and Concrete Association) remarked that like Dr Glanville he was a little worried about forms, although for rather different reasons. While the work was in progress the site had been visited by one of the most experienced American road engineers, who expressed surprise at the amount of care being taken with form setting. He commented that nothing like the same care was taken in the United States and expressed the view that the riding quality of American roads was quite satisfactory. He attributed that to the fact that in America the longitudinal float was almost invariably used for finishing, and with that machine riding quality was much less affected by the form setting than with the transverse finisher used in Britain.

Dr Collins thought, however, that the longitudinal float was not necessarily the only means of obtaining good riding quality, because from figures obtained by the Road Research Laboratory it seemed that the Danish concrete roads were at least as good as the best results ever obtained in Britain with any type of construction and yet, when seeing the Danish roads under construction recently he had been struck by the seeming lack of care given to the setting of the forms. Although he would be the last to suggest that roads should not be laid carefully, Dr Collins thought that taking more and more care was not necessarily the correct method of making progress; in fact, it was desirable, if possible, to reduce the amount of skill and care required in order to obtain good results, and he thought that an investigation into the precise effect of inaccuracies of form setting on the riding quality of concrete roads might give useful results.

Dr Collins said that he was interested in the results which had been obtained at Oxton in showing that making joints by the sawing process substantially eliminated their effects on riding quality, and that the only remaining problem seemed to be that of obtaining a good joint at the end of a day's work. The results of experimental work made by the Cement and Concrete Association on methods of sawing joints indicated that cutting at an age earlier than that used at Oxton would be preferable. If that were done granite aggregate could be cut with a silicon-carbide blade, which resulted in lower costs.

Dealing with a minor point in regard to the joint sawn with a diamond blade, Dr Collins asked whether the Author had encountered any trouble in sealing the joint.

Mr G. C. Ollerhead (Civil Engineer, A. Monk & Co. Ltd) made a few comments following the headings laid down by Dr Kirkham.

With regard to the machinery used, in Mr Ollerhead's experience the present vibrating machines of 20-ft span or more being used in Britain appeared incapable of producing a true plane surface between forms. That experience was obviously shared by the contractors at Oxton since they had utilized the expedient of placing extra weights on the finishing beam and, as shown by the profilometer readings, the results near the centre

of the bay were less satisfactory than those nearer the forms. He therefore wished to query the Author's contention that full-width construction eliminated possible causes of surface irregularities since machines of 10 to 15-ft span could be relied upon to give a truer finish between forms and should therefore give a better riding surface in the longitudinal direction. Irregularities caused by longitudinal construction joints would be parallel to the flow of traffic and should not materially affect riding quality. However, since it was undoubtedly better practice to have full-width construction, he felt that there was an urgent need for heavier equipment capable of finishing the concrete in 20-ft or greater widths to the very exacting specifications now being considered.

Dr Kirkham had suggested that irregularities in the centre of the slab were caused by the particular type of spreader employed at Oxton, but in Mr Ollerhead's experience with the hopper-type spreader, where loading was from the side of the forms, the same problem of inaccuracies along the bay centre had been encountered. It had been found that the finishing beam tended to hump upwards and thereby not completely remove the irregularities left behind the vibrating beam.

To obtain the high standard of surface finish required it was necessary to have a "workability" which allowed the elimination of surface irregularities shown up by the wet-surface profilometers. It appeared, at Oxton, that once that mix was found, further attempts to reduce the water and sand content resulted in the surface becoming unworkable. In spite of the aggregate/cement ratio being reduced to 6·25 : 1, Table 2 showed that 20% of the test beams failed, from which it could be argued that the concrete did not attain the flexual and compressive strength specified.

Would the Author agree, therefore, that in using that mix to attain the desired q value, strength had to be sacrificed to workability? Would he agree, in the light of his experience at Oxton, that more cement was necessary in the concrete to obtain fully both the required strength and riding quality?

It was noted that the top of the triangular timber fillet on the expansion joints was left $\frac{3}{4}$ in. below the surface to reduce the surge of the concrete. Was that $\frac{3}{4}$-in. dimension dictated by the maximum size of aggregate used? If $1\frac{1}{2}$-in. aggregate were used, would the fillet be left $1\frac{1}{2}$ in. down? Did the Author consider it likely that the expansion joint might close up with an increase in temperature, before the joint was sawn, thus causing the slabs adjacent to the joint to spall? Mr Ollerhead had experienced that happening in unreinforced pavements when the joint filler was left $\frac{1}{4}$ in. below the surface, with a result that the concrete spalled off up to 6 in. from the joint.

Was it a necessity for work to be stopped at expansion joints? Surely a suitable stop-end could have been put in between joints, allowing the reinforcement to run through. On a long road—say, 10 miles—that would reduce the number of stop-ends required, since being limited to 100-ft bays must reduce the length of pavement a contractor could construct in a day and indirectly, therefore, increase the cost of the road.

It was possible that the use of stop-ends, fitted flush to the top of the screeds, would have eliminated the falling away experienced when finishing up to expansion joints. He surmised that those were $\frac{3}{4}$ in. below screed level and were possibly not robust enough to withstand the full forward pressure of the vibrator without moving slightly and thus accentuating the falling away. Surge bumps, which would possibly have been left by the flush stop-end, could be removed by manual devices before a final pass with the machine.

He was interested in the construction of a 4-in.-thick lean-mix base on which to bed the screeds. Was that specified and was its primary object to fix the screed pins securely or to prevent settlement of the screeds under the load of the machines?

Mr Ollerhead had had some experience with compacted burnt shale and had found that it possessed a certain resilience; but although it generally seemed to return to original level after the passage of machines, it could not always be relied on to do so.

On the subject of formation, it appeared that some appreciable variation in level was encountered, as indicated by the varying depths of cores cut. That in itself was not conducive to good riding quality since it entailed varying adjustments of the spreader

to give the required surcharge, thus tending to disrupt the even flow of the machinery so necessary for an accurate surface.

On the subject of curing, he congratulated all concerned on their ability to prevent sun and wind crazing by spraying a water-proof membrane on to concrete laid up to 2 hours previously. Mr Ollerhead's experience had been that under adverse conditions it was essential to spray the concrete within 1 hour of placing and immediately to cover up behind with hessian covers. That meant that the concrete trains had to move forward at a rate of between 80 and 100 ft/hour, which seemed to be about double the speeds attained on the Oxton contract.

He could only assume that the differences in experiences of curing were caused by, first, the use of bag cement instead of the quicker-setting bulk cement which, under present circumstances, often arrived on site in a "hot" condition; secondly, more humid conditions prevailing, as well as the possible lower temperature in September and October than in, say, June and July; thirdly, the surface of the concrete being disturbed less owing to the smaller number of joints; and fourthly, the use of reinforcement in the slab.

Mr A. A. Osborne (Resident Engineer, Messrs Wilson & Mason) said the Author had made it quite clear that it was at expansion joints and stop-ends that the fairly high q factor had been experienced. The Author had travelled on the Continent and studied Continental methods, and it would be interesting to know what he thought of the method used in Belgium, where they had dispensed with dowel bars. Mr Osborne had always considered them essential, but now that it was necessary to construct roads at great speed and of long length it might be worthwhile, economically, to change that view.

There was now a type of expansion joint where the expansion jointing was vibrated down into the concrete to form a joint. In order that there should be no movement between two slabs in such a joint, which was usually stopped by the use of dowel bars, there was a bearing slab at the bottom of the joint. That type of construction had been tried as long ago as 1925–30 but eventually had been dropped, Mr Osborne thought correctly, in favour of dowel bars. Now that trouble was being experienced with machines, expansion joints, and stop-ends, the other system might be worth considering. What was the Author's view of that alternative type of joint?

Mr Osborne did not foresee any trouble with the stop-end, even if the work was reinforced, because the reinforcement could be cut off where it was intended to finish in, say, half-an-hour's time. As additional reinforcement a special short mat could be placed at the end of the joint to allow for any stresses arising from the absence of dowel bars. The Author had, it appeared, obtained a good result with the normal dowel joint, but why had he decided to use it and what were his views on the alternative joint? Had he considered using a slab underneath, and vibrating some suitable material down?

Why had the Author chosen slab lengths of 97 ft and 113 ft? Mr Osborne had made a rough calculation of the amount of work done per day, based on information in the Paper, and the average seemed to be about 230 ft per day. What were the average and maximum lengths of slab concreted per day?

Mr B. G. Combridge (Assistant Engineer, Surrey County Council) asked, first, whether the longitudinal joint was necessary, since several speakers had made it obvious that overcoming joint trouble was a difficult problem. Would the Author elaborate on the reasons for cutting an additional joint for a slab width of only 24 ft, apparently reinforced right across?

The Author had mentioned that he had departed from the British Standard for reinforcing mats in making the longitudinal wires 4-in. spacing instead of 3-in. spacing as specified in B.S.1221. Was that purely to get the concrete through the reinforcement more easily or in order to gain full compaction? Had the size of aggregate any influence on the spacing? Did he consider that he would be justified in future in departing from B.S.1221 or did his action foreshadow a coming amendment to that standard?

What was the method in the contract of specifying the concrete? Was it specified only by flexural or compressive strength, or was it also specified by a compaction factor?

The Author had mentioned alterations in the mixes; had those been left entirely to the contractor's discretion or imposed by the Author?

On p. 215 it was stated that the white edging strip was ramped down every 10 ft to slab level, but Fig. 2 suggested that it was dished into the slab. Mr Combridge was not clear about the method; was it tapered from ½ in. to zero thickness or was the slab underneath the edging strip dished in the manner indicated in Fig. 2?

*** **Mr T. S. Sinclair** (London Director, Robert M. Douglas (Contractors) Ltd) agreed with Dr Collins that the profilometer readings in the centre were much worse than those at the sides of the carriageway and that that would lead to the conclusion that there was deflexion and vibration in the machines themselves. Bearing in mind speakers' remarks on the excellent results obtained in Denmark and Belgium where the placing of road forms was much less exact than at Oxton, the fact that the Oxton forms were set on concrete, and also Mr Fisher's admission that those road forms were not by any means new, Mr Sinclair considered that the Author's remark "that the major fault in the formwork was the dip between the end of the forms and the first stake housing" was a particular criticism of the forms used on the Oxton job and not a criticism of forms in general. In the circumstances "initial inaccuracies in the forms" could not be objected to since the forms were, in effect, second-hand and not designed for the job on which they were being used.

With reference to conclusion (5) on p. 224, probably manufacturers would agree that any firm of repute did not send out road forms which were in any way warped. In fact, a maximum tolerance of ⅛ in. in 10 ft was the usual criterion.

It would be appreciated that road forms were made to a specification which assured maximum strength at the cheapest possible price. The type of road form used was designed to withstand the weight and vibrations of the normal machine concreting widths up to 18 or 20 ft; if wider machines were to be used, with their consequent increase in weight necessary to overcome the vibrations referred to, those who used the machines should advise the manufacturers of their new requirements and so obtain road forms which would stand up to the new weights without deflexion. No doubt the manufacturers would be pleased to give the County Councils, Ministry of Transport, and the Road Research Laboratory the forms they required—at a price!

Mr W. J. O. Scott (Principal Scientific Officer, Road Research Laboratory, D.S.I.R.) remarked that there was no doubt about the desirability of the best possible riding-surface quality on all roads, particularly trunk roads and the motor roads which were now within sight, and on which the Paper had a bearing. The fact that irregularity-index values had enabled a target to be set up and aimed at in the important work which had been described in the Paper, and that a leading representative of the road engineering industry had suggested their use as a means of attracting a bonus for improving on the target (and presumably for invoking a penalty for falling short of it), encouraged a belief that they had served, and would continue to serve, a useful purpose.

Mr Scott said he was now able to give a few additional figures which might serve to emphasize the Author's conclusions (6) and (7) in regard to joints. Considering the surface irregularity data for the four lanes in the aggregate, the overall mean value of the irregularity index q was 61 in/mile. Further analysis had shown that irregularities associated with the joints were responsible for about 11% of that total and of that proportion much the greater share, about 8%, was associated with irregularities at the stop-end joints. In justice it must be acknowledged that the introduction of a joint was liable to affect an appreciable length of the surface on either side, a length which, in the absence of the joint, would presumably have included a limited amount of irregularity, proportional to that found on the majority of the surfaces. If that length was taken as being up to 1½ ft on either side of a joint (those considerations were introduced on a

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*** This and the following contributions were received in writing upon the closure of the oral discussion.—Sec.

previous occasion[3]), it amounted to approximately 3% of the total length in that case. So although the low order of overall irregularity at Oxton emphasized the proportion associated with the joints, the disparity between 3% and 8% or 11% did underline the fact that some further improvements in joint construction technique were desirable in order to match the higher standards of surface quality which were evidently attainable on a concrete road of that type as a whole. Nevertheless, although the Author quite properly expressed his dissatisfaction with the stop-end joints, the fairly considerable interval between joints at Oxton diminished their influence so that the total irregularity associated with them was small and need give road users no cause for serious concern.

Although a number of very creditable concrete-road surfaces existed (mostly pre-war and hand-finished), it was true, as had already been mentioned, that criticism had frequently been directed at concrete roads, both post-war and pre-war, on account of unsatisfactory riding-surface quality for which the joints were usually blamed. Even though an analysis of relevant surface irregularity data had shown that in a number of post-war cases the joints were not responsible for an undue proportion of irregularity[3] (mostly cases, however, in which the overall standard of riding-surface quality was not very good), joints often attracted undue attention; they not only made an unfavourable visual impression but also an audible impression, which together led road users to infer that joints were the main source of irregularity when it was very seldom the truth. That was unjust but often unavoidable. Mr Scott suggested, therefore, that that should be countered so far as possible by efficient and tidy maintenance, including the use of modern sealing materials, the repair of spalled edges, and generally making the joints as unobtrusive as possible. Incidentally, road users almost invariably approved the lighter colour of concrete surfaces, and Mr Scott was glad for that reason that it had been decided not to darken the main concrete at Oxton, apart from the consideration that a darker colour would have engendered greater heat absorption in fine weather with consequently increased stresses.

It might be of interest to present certain considerations which arose in connexion with the possible use of longitudinal finishers such as were often employed in America, as Dr Collins had mentioned. Contours at small intervals on concrete-road surfaces tended, not unnaturally, to lie in directions roughly parallel to the length of the finishing screed, tamper, or beam. In consequence, there tended to be fewer short-length minor irregularities in a direction parallel to the length of the finishing tool than at right-angles to it. Although very good riding-surface quality had been achieved on both transversely and longitudinally finished surfaces (hand-finished), the wide variations found on different examples of both types of work precluded any valid deductions as to their relative merits. Only instances in which some work of both types had been included in the same job could afford a valid comparison, and such cases were rare; but the limited data available suggested that appreciably less overall irregularity characterized the longitudinally finished work where both types existed in comparable circumstances. Apart from the short-length minor irregularities just referred to, concrete-road surfaces might also include a number of smaller very short irregularities of small amplitude (less than 0·1 in.) which, if classified according to their shape and length, would lie in a sort of no-man's-land between the shortest genuine irregularities which gave rise to appreciable disturbing motion and the most prominent asperities which were associated with a rugous non-skid coarse surface texture. Whilst it was common knowledge that surfaces of the latter type tended to "growl" at the average modern car, those very small and minor irregularities on a concrete surface could also give rise to a "rumble" or irregular vibration, and resulted in the allegations of harsh running which were sometimes made in regard to concrete roads. In one or two cases, by virtue of the existence of cross-roads, it was possible to drive over certain slabs in directions mutually at right-angles; the usual forms of construction entailed transverse finishing and consequently vehicles normally travelled at right-angles to most of the minor contours, but when the surface was traversed in a direction

[3] Arthur Floyd, "The Trend of Road Construction" (Introductory Note), Road Division. Instn Civ. Engrs, 1950.

parallel to the minor contours (i.e., to the direction of the length of the finishing tool) a noticeable diminution of vibration and/or harshness resulted. Although the actual physical motion associated with those types of disturbance might be very slight or almost unappreciable, they were objectionable riding-surface characteristics when present. The foregoing considerations suggested that, though the resulting advantage might be less when the overall order of irregularity was low, longitudinal finishers, employed with due care, might confer a worthwhile improvement in riding-surface quality.

Mr W. P. Andrews (Roads Consultant, Cement and Concrete Association) observed that there had been little experience of concrete-road construction in recent years, though, notwithstanding the ban on the use of cement, there had been an average mileage of more than 180 during the past 5 years although very little of that work had been mechanized.

The Author had taken great care to prepare a good base on which to lay the concrete and the value of that could not be over-emphasized. Nearly all concrete road failures were caused by subgrade failures and it was interesting that the C.B.R. in that case was raised from approximately 6% to an average of well above 30%.

With regard to surface finish, whilst it was a matter of great disappointment that the q value of 40 was not obtained throughout the job, the Paper showed that it could be obtained when the engineers and the labour, both skilled and unskilled, were more familiar with the work. On the Continent, where the large scale of construction had given the necessary training and experience, q values of 40 were being obtained consistently. The Oxton job lasted only 17 days and improvement in the finish and in joint construction had been noticeable as it proceeded.

If two-course construction had been adopted better results might have been obtained. With a relatively thin top course errors in spreading and pre-compaction and ultimate finish would have been minimized, and it was interesting that that two-course construction was adopted in Holland, Germany, and Denmark, even where the road was not reinforced.

Meticulous care was taken by the contractors in the bedding of the forms, yet irregularities had occurred owing to faults in the forms themselves. A finishing machine or a finishing method which would not be so dependent on the trueness of the formwork was desirable and the German method, which eliminated forms might be considered.

An American engineer who visited the work during construction was amazed at the expense involved in the careful laying of the forms and he suggested that the American method of using a longitudinal float to finish the surface might be adopted although it was questionable if that would function satisfactorily with British very dry concrete.

Perhaps one of the most valuable features of the Paper was that it showed that the bogy of joints had been laid. The Author pointed out on p. 216 that such improvements had been made that it was virtually impossible to determine the position of the joints on the profile. The one exception was at the end of a day's run and it seemed that the difficulty there could be overcome by hand compaction, possibly using the compactor in a longitudinal direction.

Mr Andrews considered that the crossfall of 1 in 48 (see p. 203), was too steep for present-day requirements. On runways a crossfall of 1 in 80 or even 1 in 100 was accepted.

Referring to concrete strength he said the requirement of 5,000 lb/sq. in. at 28 days was too high; he thought 4,000 lb/sq. in. would be adequate. In Belgium and other Continental countries higher strengths were required, but that was where the concrete was not reinforced. A concrete strength of 4,000 lb/sq. in. reinforced was preferable to one of 6,000 or 7,000 lb/sq. in. unreinforced.

It was mentioned on p. 206 that specially made reinforcement was used which served as a tie across the longitudinal joint. It would have been more economical to use tie-bars in that case, with standard types of reinforcement.

Fig. 2c indicated that the joints were cut after 14 days. The concrete aggregate was granite and Mr Andrews considered that it should have been possible to have cut that

joint after 48 hours. In that case a silicon-carbide saw could have been used whereas with the older concrete it was necessary to use the much more expensive diamond saw. Had any difficulty been experienced in sealing the narrow cut thus obtained?

On p. 214 the Author rightly endorsed the importance of the first pass in getting good riding qualities, and that again raised the question of whether one-course or two-course work should be adopted.

The Author, in reply, said that the earlier experimental road at Oxton had been of great value, as had been shown in the Paper [4] by Mr Loe, for it had established many points in connexion with concrete road design and construction. Some parts had been designed to fail and others to last a very long time and the point between success and failure had been found. In addition, many other factors had been evaluated such as the value of reinforcement and the function of the base.

He was glad that Dr Glanville agreed with his own conclusions from the work on the Oxton extension, namely, that there was no reason why concrete roads could not be constructed with a first-class riding quality, and also that more attention to the accuracy of manufacture of forms would help considerably in obtaining that quality more easily. It seemed from Mr Fisher's and Mr Sinclair's observations on that point that at least some manufacturers were aware of the importance of it, although the Author knew of other cases within the past few years where new forms had had to be rejected because they were not straight. It seemed to him that the discussion had crystallized the essential needs in formwork: (1) that manufacturers could make straight forms to less than 1/16 in. tolerance and it was therefore to be hoped that that would become common practice with all manufacturers in future; and (2) that users of forms should handle them with greater care to ensure that no distortion occurred through rough-and-ready methods of handling. Dr Kirkham's observations on the profilometer records further emphasized the importance of accurate forms. As Mr Sinclair had stated, consideration had to be given to the ability of road forms to stand up to very heavy machines, but whatever the difficulties, the forms still had to be straight and true to line if good riding quality was to be obtained on concrete roads.

Dr Glanville had pin-pointed one of the important lessons learned from the work described in the Paper, namely, the importance of continuity of construction. It would be well worth while in future jobs, especially those of some magnitude, to give careful consideration to the layout and type of plant and to the organization of the work, having regard particularly to maintaining that continuity of construction. Thought might also be given to the overlapping of meal-breaks and to the continued presence of a maintenance man or men to deal quickly with mechanical troubles. Dr Glanville had also underlined one point which could be expressed as the slogan "Bonus for no bumps." It was unlikely that we could deal with the bonus question as easily as the Swedes but it was an idea. Mr Fisher had mentioned a measurement of the "q" value and a bonus in relation to it when considering the setting of the forms and in that respect something might be done.

The Author fully concurred with Mr Fisher's comments on the value of the detailed discussions between the engineer and the contractor on a draft specification. It ensured that full advantage was taken of the contractor's experience to meet the engineering requirements and that engendered a helpful spirit of co-operation from the very beginning. It could not be said that it would be possible to adopt that technique on an ordinary road job where a larger number of experienced contractors would normally be asked to tender, but he thought that the success of the experiment certainly demonstrated the advantages of such a procedure if it could be worked. He was grateful to Mr Fisher for the qualification he applied to Conclusion (8) in the Paper. It was quite true that the mechanical placing of the reinforcement could only be effected at present with a set-up similar to that used at Oxton. He also agreed with Mr Fisher's comments on the value of the wet-surface profilometer and was pleased to hear from Dr Kirkham that the Road Research

[4] See reference 1 on p. 202.

Laboratory was, in fact, at present considering the design of such an instrument for attachment to the commercial finishing machines.

The Author was obliged to Mr Adams for his critical examination of all the Tables and was delighted that he had found something of merit in at least one of them! It might be as well to explain the reasons why the figures of 5,000 lb/sq. in. and 600 lb/sq. in. were adopted for the minimum compressive and flexural strengths respectively. It had been the Author's original idea to use a very strong concrete, since Belgian experience had demonstrated its value in preventing cracks in relatively long unreinforced slabs. Upon reflexion, however, it seemed that the use of the richer and more workable mix involved, might present problems in mixing and placing with British machines and in British conditions which could not be completely solved during the progress of the work. Since the prime object of the experiment was to see how good a road could be built it had been thought better to use the sort of mix to which experienced contractors had been accustomed. Thus a mix of 1 : 6 to 1 : 6·5 was indicated and that would normally be expected to give an average strength of about 5,500 lb/sq. in. In preliminary trials the average strengths obtained had in fact been well over 6,000 lb/sq. in. and it had therefore been decided to raise the specified minimum crushing strength from 4,000 to 5,000 lb/sq. in. Thus, while still using a mix with which the contractor was familiar, an attempt had been made to work to higher strength figures. The strength results showed that a fair measure of success had been obtained in achieving the specified minima although the number of results falling below the specified values was larger than would normally be accepted on non-experimental work.

The difference between the proportion of results in Table 2 which did not meet the specification, and the figure quoted in the synopsis arose because the former referred to the averages for pairs of specimens and the latter to individual specimens. Of the 61 flexural specimens tested, 12 had failed to give the required strength and of the 122 half-beams tested for crushing strength 25 had failed to give the required strength. That represented 20% of failures in each case.

Regarding the average crushing strength and standard deviation, the former was lower than it had been hoped to obtain on the basis of trial mixes, namely, about 6,000 lb/sq. in. That might have been the result of using, for the trial mixes, a batch of cement which, by chance, gave higher than average strengths. The standard deviation achieved corresponded to what Sparkes[5] and Himsworth[6] had classified as "average" or "fair." It had been hoped that on the Oxton contract a value nearer 500 lb/sq. in. ("excellent" or "very good") would have been achieved and that together with an average of 6,000 lb/sq. in. would have corresponded with a minimum of about 5,000 lb/sq. in. One point which had contributed to relatively high standard deviation concerned the variations made in the mix proportions in the early part of the work, as noted in the Paper. It was thought likely that had the work lasted longer the standard deviation of the later strengths would have been considerably reduced.

Dr Kirkham's observations on the records obtained at the site helpfully amplified some of the points in the Paper. His suggested explanations for some of the small irregularities remaining, such as the uneven filling of the spreading machine, discontinuity of processes, small changes in workability of the concrete and methods of making joints, were most useful in indicating the direction in which further progress could be made. With regard to Dr Kirkham's specific suggestions about an improvement in the surface at the stop-end, it seemed a good idea to carry forward some datum on the succeeding slab as the concreting machines approached the stop-end. He could not at the moment see how it would work in practice because it should be borne in mind that the machines finishing off the slab up to the stop-end were governed in operation by the forms on which they ran and it

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[5] F. N. Sparkes, "The control of concrete quality: a review of the present position." Proc. Symposium on Mix Design and Quality Control of Concrete. C.A.C.A., London, 1954.

[6] F. R. Himsworth, "The application of statistics to concrete quality." *Loc. cit.*

was not certain that there would be time for an adjustment, as they approached the stop-end in relation to some datum set out in advance. It was an idea which might be followed up.

It might be, as Dr Collins and Mr Scott had suggested, that the use of a longitudinal float would help to overcome errors in form alignment and straightness. It was true that the Americans used that type of float successfully on their wetter concretes, as indicated by Mr Andrews, but it was doubtful whether it would be sufficiently powerful to improve the surface of the dry concretes used in Britain and might in any case cause some tearing in the surface. That was another idea which could easily be tried out on an actual construction job and lent point to the expressed views of the Author and others that if only engineers were given the opportunity to build some concrete roads, most of the problems would be quickly solved. In the absence of special equipment such as a longitudinal float, the Author could not agree with Dr Collins that too much care was being taken in setting forms. No doubt the contractors on the Oxton job spent a lot of time and energy in setting the forms to the best of their ability and if forms were always manufactured to the high standard referred to by Mr Fisher a lot of that time and energy could be saved, but, having regard to the very high standard which should be set, meticulous care should be given to form setting. That was the datum for all machine operations.

He had seen the records of measurements with the bump integrator in Denmark and had been amazed by the results produced, having regard to a description of their methods; he hoped that Dr Glanville and the Road Research Laboratory engineers would be able to make further investigations. Was it possible to analyse the Danish methods in some way to find out how they achieved relatively good riding qualities, notwithstanding their apparent indifference in form setting?

Dr Collins had asked whether there was any difficulty in sealing the groove cut by the diamond saw. The Author had watched it closely and spent a lot of time with those doing it, and to his surprise the joint had been perfectly filled. The filler was heated to such a temperature that it flowed evenly and the man had become expert in the job. One could see the filler running along the bottom of the joint in advance of the pouring. That was the method adopted: so long as the filler could be seen flowing in the bottom of the joint in advance then the joint was perfectly sealed.

The first point raised by Mr Ollerhead concerned the effect of full-width construction on riding quality. It might be true, as he had remarked, that the use of half-width construction facilitated the production of a plane surface and that would certainly apply in the first lane. The Author felt, however, that in constructing the second lane, any irregularities in the surface of the first lane (on which the machines would run) would be reproduced in the second lane and added to those irregularities normally produced. It seemed likely therefore that a relatively poor riding surface would be obtained in the second lane. He fully agreed with Mr Ollerhead that there was an urgent need for heavier equipment capable of finishing very wide slabs to the present exacting specifications. There would seem to be no doubt that such machinery could and would be produced when the demand arose and the experience at Oxton had demonstrated its importance. It might be pertinent to note that runway specifications were now much more stringent than formerly in regard to riding quality and, with the large amount of work in hand, the necessary machines might be developed for that purpose and subsequently used in full-width road construction. Dr Kirkham [7] had previously described the advantages and disadvantages of the different types of spreading machines. The spreader used at Oxton was of a new kind developed by the contractors and was considered by the Author and others to be better than any spreader used hitherto. Whilst it was probably correct that differences occurred in the precompaction of the concrete across the width of the bay, that disadvantage was not, in the Author's view, as serious as the variation in precompaction obtained with orthodox spreading machines.

The Author did not agree with the suggestion that strength should be sacrificed to workability to achieve the desired riding quality. If the workability were too high, it

[7] See reference 2 on p. 205.

would be difficult to control variable flow of the concrete under the vibrating beam on slab having a gradient or crossfall: that would result in poorer riding quality. If the cement content were increased there was the greater possibility of laitance being produced on the surface which might give both a slippery surface and scaling from frost action. It was noticeable that in Belgium, where a mix of about 1 : 5 was used, parts of the Ostend Brussels motor road were labelled "Danger de glisser". Also as referred to previously, in replying to Mr Adams, it should be noted that the target figures for strength had been raised by about 25% above what was normally considered adequate. The Author was grateful to Mr Ollerhead for referring to the possibility of spalling at the expansion joint before sawing the groove at the top. It had not happened at Oxton, where the temperature had been rather high throughout the work but it was easy to visualize that such defect might arise should the concrete be subjected to a large rise in temperature between the times of laying the concrete and sawing the joint.

The Author did not view with favour the inclusion of construction joints at the end of day's work, to avoid finishing always at an expansion joint. It seemed to him that that would introduce an additional cause of irregularity since there was almost bound to be a bump at such a joint. It should be noted also that the contractor had had some latitude in organizing his day's work since the slab lengths had been specified to be between 97 and 113 ft, and if he had been doubtful of completing a given number of slabs of the longer length he could reduce his slab length to the lower value. The Author could assure Mr Ollerhead that the stop ends had always been right up to the finished level: the irregularity had been formed not at the actual joint itself but some little distance back and that had been due to lack of forward movement of the machine. It was the secondary effect which in some cases was more serious than the difficulty at the stop-end itself. The 4-in.-thick lean-concrete base on which the forms had been bedded had been provided to ensure full support for the forms. It was possible that the compacted colliery shale might, itself, have been strong enough but it was thought better at Oxton to be quite satisfied that forms were fully supported. Mr Ollerhead was not correct in deducing from the data on cores, that there were considerable variations in level of the colliery shale base. The figures for the heights of the cores were normally subject to rather wide variations owing to quite small and local depressions or bumps—the sort of minor defect which could easily occur between rolling the base and laying the concrete, especially if the men were allowed to walk or wheel barrows over the prepared formation. It was of interest to note that the average value of the heights given in the Table was 8·2 in., which was quite close to the specified value.

In reply to Mr Osborne, the Author considered it would be a mistake to omit the dowell bars at expansion joints. Experience on motor roads in Germany and Belgium had shown that where dowel-bars had been omitted in roads carrying heavy traffic, serious faulting sometimes occurred. Some slabs on the German motor roads, for example, were up to 1 in. out of vertical alignment on each side of a joint. It was true that the sleeper beam referred to by Mr Osborne, was used in Holland and, at any rate after a few years, appeared to be satisfactory. In view of pre-war experience in Britain, however, the Author did not feel that they were a satisfactory answer to the problem of load transfer; they were relatively useless, of course, when the slabs were warping upwards at the ends and their insertion caused a disturbance in the carefully prepared base just in the areas where the stability of the base was very important. Before substituting sleeper beams for dowel-bars, the effect of the Dutch type should be observed after a longer period under traffic. It was only fair also, to point out that the construction of dowelled expansion joints during the day's run did not prevent the production of a first-class riding quality in their vicinity. The only objection to dowel-bars that the Author could see was in preventing the use of the vibrating plate to form the expansion joint in the freshly laid concrete. In his view the advantages of effective load-transfer under all conditions of warping, freedom from subsequent differential settlement, and uniformity in consolidation of the base more than outweighed the passing advantage of allowing the use of a special machine for their formation.

Mr Osborne had raised a point which the Author had fully expected—the length of the slab. The slab-lengths ranged from 97 to 113 ft with an average of about 100 ft. The idea had been to have an irregular length, in case some rhythmic disturbance might be created, and 100 ft had been selected after certain observations on the Continent, after discussion with the Germans on the length they were using, and after consideration of the desirability of minimizing the number of joints, of the decision to eliminate contraction joints, of the whole design of the slab, thickness, weight, reinforcement, and so on; and the conclusion had been reached that 100 ft, more or less, was a reasonably safe unit to adopt with little possibility of cracking.

As for the output on the job, the machines had not been tested to full capacity for various reasons; on some days two slabs, each about 100 ft, had been laid and on some days three slabs, but so far as he could recollect, there had never been more than three slabs laid in one day.

Mr Combridge had asked why a longitudinal joint had been considered necessary in view of the troubles known to be attendant upon joints generally. The Author replied that a longitudinal joint was very desirable with a view to minimizing the possibility of longitudinal cracking, having regard to the comparatively long bay adopted. No irregularity whatever had been introduced by the cutting of that joint and although there had been a small expense and the question of the machine operation, he did not look upon that as trouble and felt that there was no trouble at all with surface irregularity.

On the question of the mesh of the reinforcing fabric, the spacing of the longitudinal bars had been increased to 4 in. instead of the 3 in. in B.S.1221, since it had been thought that that might assist in getting good compaction of the concrete. It was possible, of course, that a spacing of 3 in. would have been satisfactory and, presumably, if any change were contemplated in the Standard, that would only be made as a result of more experience than had been obtained at Oxton and possibly the carrying-out of a comprehensive experiment. The concrete itself had been specified in relation to minimum compressive and flexural strengths and the workability of the concrete had been based on contractor's experience. The amendments to the mix proportions had been made by agreement with the contractor.

The white edging strip had been ramped down so far as the surface was concerned but not floated out to a feather edge. There was a thickness of concrete underneath. The man doing the work had scratched out rather more of the concrete there than elsewhere, the whole idea being to get a proper bond between the white edging strip and the concrete slab and to have a reasonable thickness the whole way and not work to a feathered edge, which might well spall off.

The Author was grateful to Mr Scott for his amplification of the results of the multi-wheel profilometer records. It was very useful to have the factual confirmation of many of the impressions gained as a result of examining the early records. Mr Scott's views on the importance of neat and tidy maintenance of joints were apt. It seemed a fact that because concrete roads required a very small amount of maintenance they often received none at all, which sometimes led to the development of faults which were easily and cheaply preventable.

The question of two-course construction advocated by Mr Andrews was a debatable one, and the fact that it had been adopted in Holland, Germany, and Denmark did not necessarily mean that it contributed in major degree to a good riding quality. So far as the Author knew, Denmark was the only country which had produced better results than at Oxton, and the Danish roads were designed without reinforcement and with very few, if any, expansion joints—both factors which would help considerably in improving the surface quality. The use of two-course construction would, of course, involve a second train of machines and the Author felt that better comparisons between single- and two-course work were required before the latter could be recommended for general adoption. With regard to crossfall, there was a tendency to decrease the amount of crossfall with the introduction of smoother running surfaces; a very good reason for not going below 1 in 48 was that the steeper slope allowed the water to get away reasonably readily to the channels,

particularly during periods of alternate frost, snow and thaw, and, as a practising highw engineer with responsibilities beyond initial construction, the Author could not consid adopting a flatter crossfall. He did not agree with Mr Andrews that the provision of t bars across the longitudinal joint would have been more economical than the use of fab with increased transverse reinforcement. The reinforcement manufacturers could readi change the wire-size and spacing of their welded mesh fabrics, whereas the inclusion of t tie-bars would have involved considerable difficulty in placing and fixing especially wh the paver was operating between the forms. The delay of 14 days in joint cutting suit the convenience of the contractors since the experienced workmen necessary for setti out and cutting the joints had been engaged in other work during concreting. Schönben had stated that: "The operation of cutting the joint in the concrete when it is only thr days old, oddly enough takes more time than cutting a joint in older concrete". TH was at variance with Mr Andrews's comment and it might well be as he had implied, th the difference might be due to the use of different types of saw-blade. Any informati Mr Andrews might have on that subject would be very welcome and helpful to engine if it could be published.

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[8] W. Schönberg, "Beobachtungen bei dem Bau von Betonfahrbahndecken". Stras u. Autobahn, vol. 8, 1955, p. 278.